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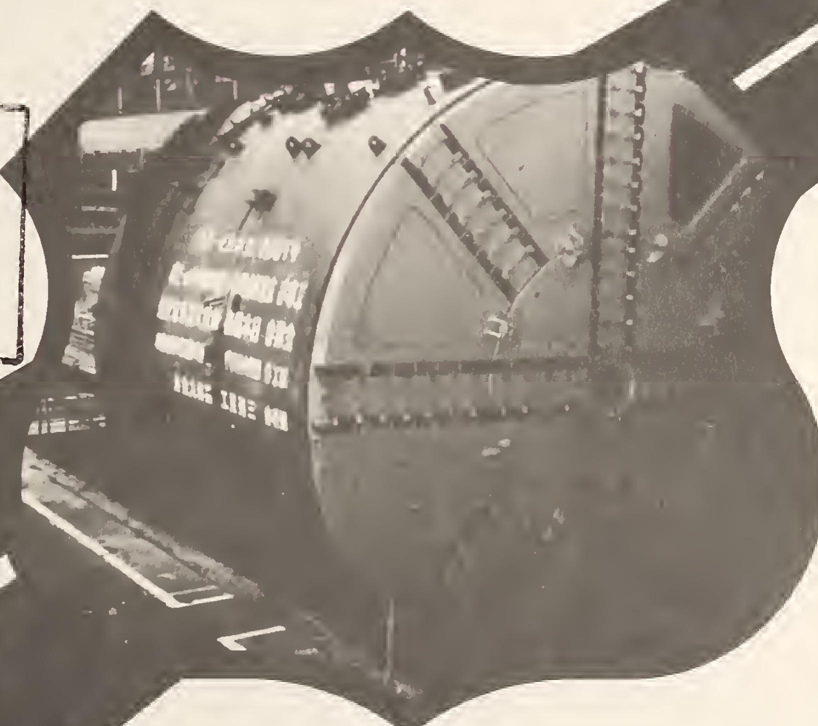
ROUNDWATER CONTROL IN TUNNELING

Vol. 3. Recommended Practice
April 1982
Final Report



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Federal Highway Administration

Offices of Research & Development
Structures and Applied Mechanics Division
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FOREWORD

This volume summarizes the authors' recommendations for controlling groundwater during underground construction and thereafter during the life of the tunnel. The use of example problems typical of a broad range of site and tunnel situations emphasizes the applicability of the alternate control methods.

Sufficient copies of the report are being distributed to provide two copies to each regional office, one copy to each division office, and two copies to each State highway agency. Direct distribution is being made to the division offices.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

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16. Abstract

This volume summarizes Volumes 1 and 2 and presents guidelines for recommended best practice in a concise format. Design and construction details not included in the more general descriptive nature of Volumes 1 and 2 are included herein.

The final chapter of this volume is a discussion and recommendations for possible implementation of innovative methods and for further research.

Three companion volumes include:

Volume 1: Groundwater Control Systems for Urban Tunneling (FHWA/RD-81/073)

Volume 2: Preventing Groundwater Intrusion into Completed Transportation Tunnels (FHWA/RD-81/074)

Executive Summary (FHWA/RD-81/076)

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PREFACE

This is Volume 3 of a three-volume series devoted to groundwater control during and after construction of transportation tunnels in urban areas. The work presented herein is a compilation of direct experience of the principal authors, along with that of selected contractors and consultants familiar with groundwater control methods.

Volume 1 discusses methods for controlling groundwater during construction of tunnels.

Volume 2 discusses methods for controlling groundwater during the life of the tunnel.

Volume 3 is a compilation of Volumes 1 and 2 into a concise format, including example problems plus recommendations for future investigations.

The work has been sponsored by the Federal Highway Administration of the U.S. Department of Transportation, Office of Research Structures and Applied Mechanics Division.

Individuals outside of the performing organizations who gave freely of their time and expertise included:

Mr. Robert S. Mayo	- Robert S. Mayo & Associates, Washington, D.C.
Mr. Richard Redmond	- MacLean Grove & Company, Greenwich, Connecticut
Mr. Joseph Bhore	- Perini Corporation, Framingham, Massachusetts
Mr. John A. Shuster	- Geofreeze, Lorton, Virginia
Mr. Robert M. Snyder	- Tekken Construction Co. Ltd., Walnut Creek, California
Mr. Joseph Welch	- Hayward Baker Co., Odenton, Maryland

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APPLICABLE SI UNIT CONVERSIONS

AREA

1 acre	= 4047 sq. m.
1 sq. in.	= 6.45 sq. cm.
1 sq. ft.	= 0.0929 sq. m.
1 sq. mi.	= 2.59 sq. km.

DENSITY

1 lb. mass/cu. ft.	= 16.018 kg/cu. m.

FLOW

1 gallon/min.	= 0.063 l./sec.
1 gallon/min.	= 0.00379 cu. m./min.

FORCE

1 lb. - force	= 4.448 Newtons
1 kg. - force	= 9.807 Newtons

LENGTH

1 in.	= 25.4 mm.
1 ft.	= 0.3048 m.
1 yd.	= 0.9144 m.
1 mi.	= 1.609 km.
1 mil	= 0.0254 mm.

PRESSURE AND STRESS

1 lb. per sq. in.	= 6.895×10^3 Pa.
1 atm.	= 1.013×10^5 Pa.
1 kg-force/sq. cm.	= 9.807×10^4 Pa.
1 bar	= 1×10^5 Pa.
1 kip/sq. in.	= 6.895×10^6 Pa.
1 lb-force/sq. ft.	= 47.88 Pa.

VOLUME

1 cu. in.	= 16.4 cu. cm.
1 cu. ft.	= 0.0283 cu. m.
1 cu. yd.	= 0.765 cu. m.
1 gal.	= 0.00379 cu. m.

1.00 INTRODUCTION

This is Volume 3, entitled "Recommended Practice," of a four-volume report of Groundwater Control in Tunneling.

The other volumes include:

Volume 1: Groundwater Control Systems for Urban Tunneling

Volume 2: Preventing Groundwater Intrusion into Completed Transportation Tunnels

Executive Summary

In addition to being a condensation of key information included in Volumes 1 and 2, this third document contains a series of examples in Section 4.90 which illustrate application of various groundwater control methods during construction as well as design and construction details not included in the more general descriptive nature of Volumes 1 and 2.

Section 9.00 presents recommendations for future work on groundwater control methods both during and after construction of transportation tunnels.

Reference numbers cited in Sections 4.00 and 6.00 are found in Volume 1 and those cited in Section 5.00 and 7.00 are found in Volume 2.

2.00 EVALUATION OF SUBSURFACE CONDITIONS

2.10 GENERAL

Improper control of groundwater is probably the greatest hazard faced by tunnelers, yet subsurface exploration programs undertaken for design of tunnels frequently consider groundwater as a factor of secondary interest. Evaluation of subsurface conditions pertinent to groundwater control is usually a multi-phased process as follows:

Phase 1: Preliminary Investigations

Phase 2: Design Phase Investigations

Phase 3: Construction Observation

The Preliminary and Design phases are the truly investigatory phases of the work. Major elements of these programs are summarized in Table 1.

The Construction Observaton phase, by comparison, is not an investigation per se. It is a fine tuning of the pre-construction information based on observed conditions during construction.

2.20 PRELIMINARY PHASE

The preliminary Phase includes collection of all available data from a variety of sources. Table 2 is a listing of commonly referenced sources of subsurface data. It may also include a program of widely spaced borings to identify major geologic conditions. Piezometers or observation wells should be installed in all borings in which groundwater is encountered or is anticipated. These groundwater observation points should be monitored continuously through the design and construction period as well as any lapses in between project phases in order to permit development of seasonal groundwater data.

Limited laboratory testing of soil samples may be undertaken for proper soil classification, but it is not essential to the preliminary program. Laboratory testing of groundwater samples if any, is normally undertaken at this time only if unusual conditions are suspected such as high sulphate or hydrogen sulphide content, the presence of unusual industrial waste, etc.

Preliminary data, while often lacking in detail, are very significant because project feasibility and early cost estimates are often based on this information.

TABLE 1. SUMMARY OF SITE INVESTIGATION TECHNIQUES FOR EVALUATION OF GROUNDWATER CONTROL

WORK PHASE	GEOLOGIC CONDITIONS	EXPLORATORY PROGRAM GUIDELINES							NOTES
		BORINGS AND WATER OBSERVATIONS				LABORATORY TESTS		SPECIAL TESTS	
		SPACING	DEPTH	TYPE	O.W./Pz.	SOIL/ROCK	WATER		
PRELIMINARY	SOIL	1,000 ft. (305m.) to 2,000 ft. (610m.)	A minimum of 2 tunnel diameters below probable invert	2 1/2 in. (6.4 cm.) dia wash boring, split spoon samples. ----------- Continuous Bx size rock core	In every completed boring (See Note 3) if ground-water is encountered or expected to be encoun-tered.	Limited number of soil class-ification tests. ----------- Engineering properties can be approximated from Alter-berg limit data.	Use avail-able data. If none available, a limited number of tests may be required for evalu-ation of corrosion and encrus-tation potentials.	Usually none required.	1. Research all existing data. See Table 2 for sources of infor-mation. 2. Full-time qualified observation of explorations is essential. 3. Observation wells with surface seals are normally ade-quate. If perched water or a confined aquifer is sus-pected, install piezometers in critical horizons.
	ROCK	See above	See above	3 1/2 in. (8.9cm) diameter wash boring in soil, split spoon samples. ---------- Continuous NX size rock core	See above	Usually no laboratory tests of rock cores required.	See above	See above	

TABLE 1. SUMMARY OF SITE INVESTIGATION TECHNIQUES FOR EVALUATION OF GROUNDWATER CONTROL (continued) 2/3

WORK PHASE	GEOLOGIC CONDITION	EXPLORATORY PROGRAM GUIDELINES							NOTES
		BORINGS AND WATER OBSERVATIONS				LABORATORY TESTS		SPECIAL TESTS	
		SPACING	DEPTH	TYPE	O.W./Pz.	SOIL/ROCK	WATER		
DESIGN	SOIL AND ROCK	300 ft. (91.5m.) to 500 ft. (152.5m.) (See Note 3)	A minimum of one tunnel diameter below probable invert.	• 3 1/2 in. (8.9 cm.) dia. wash boring. split-spoon samples. (See Notes 1 & 2) • Continuous NX size core in rock.	• Installed in completed borings at spacings on the order of 1,000 ft. (305m.) • A minimum of two at each station or major underground chamber.	• Classification tests of every major soil unit. • Engineering property tests of fine grained soft compressible soils. • Certain sedimentary rock susceptible to solution may require mineralogic testing to identify soluble components, i.e., carbonates	• Total hardness • Total Fe • Total Ng • Alkalinity • Chlorides • Sulphates • Nitrates • pH • Hydrogen sulphide • Carbon dioxide • Dissolved oxygen • Total dissolved solids • Silica	• Falling and constant head borehole permeability tests in soil to identify major differences in soil permeability. Special care required in test interpretation. • Packer tests in rock. Limited objective pumping tests. • Long-term pumping tests to investigate significant aquifers. • Special performance tests to evaluate innovative groundwater control techniques. • Geophysical techniques may prove helpful in augmentation of boring program, but should be used with particular care in urban areas.	1. Continuous soil sampling advisable at tunnel depth to identify stratification. 2. Undisturbed samples of soft compressible soils should be obtained from each major soil unit. 3. Closer spacing on the order of soft 50 ft. (15.m) to 200 ft. (61.0m.) may be required in station areas. 4. Full-time qualified field observation of explorations is essential.

TABLE 1. SUMMARY OF SITE INVESTIGATION TECHNIQUES FOR EVALUATION OF GROUNDWATER CONTROL (continued) 3/3

WORK PHASE	GEOLOGIC CONDITION	EXPLORATORY PROGRAM GUIDELINES							NOTES
		BORINGS AND WATER OBSERVATIONS				LABORATORY TESTS		SPECIAL TESTS	
		SPACING	DEPTH	TYPE	O.W./Pz.	SOIL/ROCK	WATER		
DESIGN	SOIL AND ROCK							• If unusual water quality parameters are measured, special treatment tests may be necessary. • If water is discharged into surface water, special tests may be required to evaluate effect of discharge water on aquatic life in receiving waters.	

TABLE 2. SUMMARY OF MAJOR SOURCES OF AVAILABLE GEOTECHNICAL DATA

Published Data

1. U.S.G.S. Surficial Geology Maps
2. U.S.G.S. Bedrock Geology Maps
3. U.S.G.S. Hydrological Atlases
4. U.S.G.S. Basic Data Reports
5. State and County Geologic and Hydrologic maps and reports.
6. National and Local Technical Journals, Magazines and Conference Proceedings.
7. U.S.S.C.S. Soil Maps

Unpublished Data

1. Local test boring and well drilling firms
2. Local and State highway departments
3. Local water departments
4. State Well permit records
5. State and Local transportation departments
6. State and Federal Environmental Agencies
7. State and Federal Mining Agencies
8. Army Corps of Engineers
9. Local consulting, construction and mining companies
10. Geologists, Hydroleogists, and Engineers at local universities
11. Historical records
12. Interviews

Notes: U.S.G.S. - United States Geological Survey
U.S.S.C.S. - United States Soil Conservation Service

2.30 DESIGN PHASE

The project design phase is typically done in multiple stages of progressively increasing detail. The primary source of information is typically a well documented test boring program. Experienced field supervision and documentation is essential. Piezometers and/or observation wells should be installed in a representative number of the completed borings for proper definition of the groundwater regime. It is suggested as a first approximation that groundwater monitoring points be established at an interval not to exceed 1,000 feet (305 m) along the alignment. Actual spacing for any particular project will vary depending on the geohydrologic setting, i.e. soil or rock permeability, location of barrier boundaries, recharge sources, groundwater divides, or surface conditions which could limit access to instrument locations.

Geophysical surveys may be performed to supplement boring information, but for design of transportation tunnels which are typically at relatively shallow depths, they are of secondary importance to boring and piezometeric data. Tables 3 and 4 summarize the more common geophysical techniques currently in use for evaluating general subsurface conditions from the ground surface and from borings, respectively.

Borehole permeability testing is useful to locate strata with major differences in permeability, but there are many factors which render these data questionable. Of greater value in evaluation of field permeability is proper sample classification and limited objective pumping tests. Reference 139 contains a thorough discussion of borehole permeability test methods.

Packer tests in rock are similar to borehole permeability tests in that they can be useful to locate zones of major differences in permeability. Borehole permeability testing, whether in soil or rock, should not be relied upon without other confirmatory laboratory and field information. Refer to Reference 277 for discussion of test details.

For evaluation of aquifer permeability, probably the best laboratory tests are simple grain size analyses. Other tests may be necessary if soft compressible soils are present. Such tests include Atterberg Limit determinations, compressibility, and shear strength to help in the evaluation of possible effects of dewatering (such as settlement and lateral movement). A more complete discussion of laboratory testing is included in Section 3.40 of Volume 1.

TABLE 3. COMMON GEOPHYSICAL METHODS EMPLOYED FROM THE GROUND SURFACE FOR GROUNDWATER STUDIES
(Extract, Ref. 235)

METHOD	EFFECTIVE DEPTH RANGE	BRIEF DESCRIPTION OF TECHNIQUE	APPLICATIONS	PARAMETER MEASURED	PARAMETER INFERRED	ACCURACY OF INFERENCE	COMMENTS
Seismic Refraction	0-200 ft. + (typical)	Seismic impulse introduced at or near ground surface, impulse transit time to a linear array of geophones measured, pattern of transit times interpreted to determine subsurface velocity units, unit thicknesses, and attitudes.	Mapping of subsurface soil/water table/bedrock velocities, depths, and thicknesses. Materials classification.	Transit times of elastic waves	Apparent horizontal velocity (V) interface depths (D)	±5% x V ±10-15% x D	Survey depths approximately one-third of maximum source-geophone distance, resolution of layers limited by seismic wave length/velocity, and density contracts. Calibration by observation in boreholes improves accuracies significantly.
Seismic Reflection	200 ft. +	Seismic impulse introduced at or near ground surface, impulse transit times from surface to subsurface reflector to surface recorded and measured at surface geophone positions, pattern of transit times interpreted to determine subsurface velocity units, unit thicknesses and attitudes.	Mapping of subsurface soil/water table/bedrock velocities, depths, and thicknesses.	Transit times of elastic waves	Apparent vertical velocity (V) interface depths (D) attitudes of interfaces	±5% x V ±5% x D ±10°	Shallow surveys subject to direct/refracted signal interference. Resolution of layers limited by seismic wave length/velocity and density contrasts. Calibration by direct measurement in borehole improves accuracies significantly.
Resistivity/ Conductivity	No Limits	Constant or slowly varying DC introduced into ground by two powered electrodes, pattern of voltages is converted to apparent resistivity pattern, resistivity pattern interpreted to determine subsurface apparent resistivity units and unit thicknesses. A wide variety of electrode positioning is used.	Mapping subsurface of soil/water table/ bedrock resistivity units. Rapid areal mapping of strong subsurface resistivity contrasts, grounding potential for high voltage operations.	Induced voltage (V)	Apparent resistance (ohms)	±2-5%	Theoretical penetration depth ranges from 0.1 to 0.4 times power-receiver electrode separation (depends upon configuration of electrode placements).
Electro-magnetic	0-50 ft. (maximum)	Electromagnetic energy pulses introduced into the subsurface in a narrow beam, reflected energy detected very near source, pattern and strength of signal from reflectors in subsurface interpreted to determine depth to reflecting horizons or objects.	Mapping of subsurface reflectors, location of anomalous subsurface discontinuities.	Reflected EM pulse (transit time, amplitudes)	See Comments	variable	Amplitude of reflected pulse is recorded in shades of gray inferring interface dielectric contrasts. Wet clay layers tend to disperse signal and limit penetration depth. Interface depths a function of pulse velocities, pulse velocities variable.

1 ft. = 0.3048 in.

TABLE 4. COMMON GEOPHYSICAL METHODS EMPLOYED IN BOREHOLES FOR GROUNDWATER STUDIES

(Extract, Ref. 235)

LOGGING METHOD	BRIEF DESCRIPTION	APPLICATIONS	PARAMETER MEASURED	BOREHOLE CONDITION	LOGGING RATES	COMMENTS
Sonic/Acoustic	Pulsed transmitter in borehole tool emits sound waves that propagate through borehole fluids and sidewall strata, transmitted wave arrivals detected by transducer on tool, transmit times of elastic waves interpreted from continuous log of borehole response.	Continuous subsurface seismic wave velocity profile, velocity contrast locations, inferred engineering parameters from wave velocities, fracture zone identification and location.	Transmit time of elastic waves, relative wave amplitudes, depth from cable length and tool measurement.	Fluid-filled cased/uncased	30-100 ft/min	Eccentered tool for dry boreholes under evaluation, hole-to-hole surveys (~50 ft. separation) experimental. High Void Ratio materials (VR >0.3) limit usefulness in soft ground.
Seismic	Seismic waves initiated by impulsive source (explosives common), transmit times to surface detectors (uphole survey) or detectors in other boreholes (crosshole velocity surveys) analyzed to obtain compressional and shear wave velocities.	Semi-continuous subsurface velocity profile, inferred engineering parameters from seismic wave velocities.	Transit time of elastic waves, depth from cable length.	Fluid-filled/dry, cased/uncased, common	~ 1000 ft/day	Has relatively wide use in engineering community; analytical difficulties in identification of direct shear wave because of interference by direct or converted compressional waves, and in identifying actual wave paths.
Thermometric	Borehole traversed by thermister or other temperature sensor, absolute or relative temperature recorded, temperature gradient measurement common. Based upon change of resistance as a function of temperature for sensor materials.	Location of water table, inflow-outflow zones, casing anomalies, grouting levels, outside casing.	Change of resistance with temperature, depth of change (cable length).	Any	~ 100 ft/min	Provides corrections needed for other logs if temperature variations are extreme or absolute temperatures high (>150°C).
Visual/ Imagery	Borehole traversed by camera or ultrasonic transmitting/receiver, tool, direct photographs or high resolution images from reflected acoustic waves obtained from logging run.	Semi-continuous or continuous "picture" of borehole wall conditions to identify stratigraphy, stratigraphic changes, and physical appearance to infer stability, fractures shear zones, gross grain size distribution, permeable zones, etc.	Change in visual appearance or change in acoustic reflectance, depths from cable length.	Any, dry or clear fluids most common.	~100 ft/min	Most uncased boreholes require flushing with clear water to use photography or television tools in saturated zones, or mud conditioning for ultrasonic tools.
Normal Resistivity	Two or more electrodes lowered into borehole, one powered and other(s) as sensing electrodes with typical: spacing 16", 64", and 15 ft+ (short, long, and lateral, respectively). Logging consists of a continuous recording of voltage variations at sensing electrodes caused by resistance changes in sidewall strata.	Continuous subsurface profile of electrical potential in borehole inferring stratigraphy, stratigraphic changes, porosity, permeability, fluid conductivity, bulk density.	Potential (voltage) at each sensor electrode, depth from cable lengths.	Fluid-filled, uncased.	~100 ft/min	Theoretical average sidewall penetration 0.25 - 0.50 times electrode spacing (influenced by actual electrical resistivities and stratigraphic sequence), corrections for borehole conditions, strata thicknesses, fluid resistivities, and borehole fluid migration into sidewall are required.

1 ft. = 0.3048 in.

In addition to investigation of the hydraulic nature of an aquifer, it is important that groundwater quality be studied. Water quality has important influences on selection of equipment, i.e., potential corrosion and encrustation problems, and on the environmental concerns relative to water disposal. Table 5 is a summary of major water characteristics affecting groundwater control systems.

Special investigations conducted as part of the design phase work are normally made following thorough study of all conventional data, i.e., test borings, geophysics and laboratory data. Special investigations may include long term pumping test, full scale test sections, and extensive environmental analyses such as bio-assay testing to determine tolerance of plant and aquatic life to proposed methods.

2.40 CONSTRUCTION OBSERVATION PHASE

The third phase of any subsurface investigation relative to groundwater control is construction observation. Subsurface investigations do not end when the contract is awarded. Data are continually developed during the installation and monitoring phases which must be reviewed to permit system modification as necessary.

The number and spacing of dewatering extraction devices can be varied from original pre-construction designs during the installation process. Test pumping of the early wells, wellpoints or ejectors, for instance, will indicate if the anticipated spacing is sufficient to achieve the desired results.

Similarly, the need for recharge can be determined during construction based on observed water levels and ground movements. Table 6 is a summary of common monitoring devices used for measuring ground behavior.

TABLE 5. SUMMARY OF GROUNDWATER CONSTITUENTS AFFECTING DEWATERING SYSTEMS

CONSTITUENT	EFFECT
Calcium Carbonate	--Encrustation of screws and surrounding materials due to precipitation as amount of dissolved carbon dioxide decrease with decreased pressures.
Iron Compounds (>0.5 ppm)	--Encrustation as with calcium --Iron hydroxide deposited as jelly-like substance due to increased velocities. --Black ferrous oxide, red ferric oxide and white ferrous hydroxide form due to increased velocity and exposure to oxygen in dewatered voids. --If water contains less iron than its capable of sever corrosion can result.
Manganese Compounds	--Similar to iron
Bacteria	--Iron bacteria feed on carbon compounds including bi-carbonates and carbon dioxide producing a well clogging slime.
Hydrogen Sulphide	--Corrodes steel and copper based alloys. Copper sulphide encrustation can result.
Carbon Dioxide	--Dissolved carbon dioxide forms carbonic acid which is corrosive.
Oxygen	--Dissolved oxygen accelerates corrosion.
Total Dissolved Solids	--Accelerates corrosion due to increased electrical conductivity.
Hydrogen Ions	--A pH less than 7 results in an acid solution which is corrosive.
Silica	--Will combine with iron and manganese to form encrusting silicates which are insoluble to acid.

TABLE 6. SUMMARY OF GROUNDWATER CONTROL MONITORING INSTRUMENTS

	Instrument	Use
1.	Piezometers and Observation Wells	- Measurement of piezometric levels associated with all groundwater control methods. (See Figure 3)
2.	Surficial Settlement Measurement	- Measurement of surface settlement resulting from groundwater control. Usually monitored by optical survey, but can be done with water level devices and pneumatic settlement systems.
3.	Deep Settlement Devices	- Measurement of settlement at depth. Can be a single deep anchor device (Figure 8) or a multi-point system (Figure 9).
4.	Lateral Movement Devices	- Measurement of lateral movement adjacent to open cuts and bored tunnels in soil and rock (Figure 10).
5.	Thermocouples/ Thermistors	- Measurement of temperature in the ground for evaluation of freezing processes (Figure 30).
6.	Accoustic Emission Monitors	- Measurement of grout travel in soil and rock. A method still in research.
7.	Resistivity	Measurement of chemical grout travel in soil. A method still in research.

3.00 METHOD SELECTION

3.10 GENERAL

Until recently the selection of dewatering systems has largely been made without the benefit of detailed analytical geohydrologic studies. This practice now appears to be changing. Several of the factors bringing about this change are discussed below.

One possible reason is a growing recognition that groundwater is a valuable, limited resource. The need for aquifer protection and groundwater management has become clearly evident. Added awareness of available analytical methods is filtering into the construction industry and is expected to result in their common usage for civil construction applications.

A second reason is the expanding capabilities of both programmable calculators and microcomputers. Hand-held programmable calculators can be readily programmed to solve the well function, making the evaluation of problems by image well theory a far less time consuming task. Aquifer modeling by use of finite difference or finite element techniques is now practical on desk-top microcomputers. Numerical modeling technique are rapidly replacing closed form solutions and graphical flow net solutions in all but the simplest cases. White flow nets are still a valuable analytical tool, they can be drawn quicker and more accurately by a computer than by hand.

A third reason for the increased use of analytical methods is the rising cost of energy. Recognizing that the operating costs of a system will vary directly with efficiency, there is now an added incentive to develop more effective systems. However, it must be recognized that the cost of the system will be controlled in part by the availability of construction equipment and in part by local union practices. The final selection of a dewatering system is therefore likely to remain somewhat of an art for the foreseeable future.

Key elements in the process of selecting an appropriate groundwater control method for use druing construction is described in Section 3.00 of Volume 1 in generalized descriptive terms. In this volume the selection process has been systemitized and is presented in a series of matrices. Figure 1 is a flow chart which illustrates the process followed by a series of selection matrices (Figures 2 through 6). A series of example problems are included in Section 4.90 which illustrate how selection of technically feasible methods is accomplished.

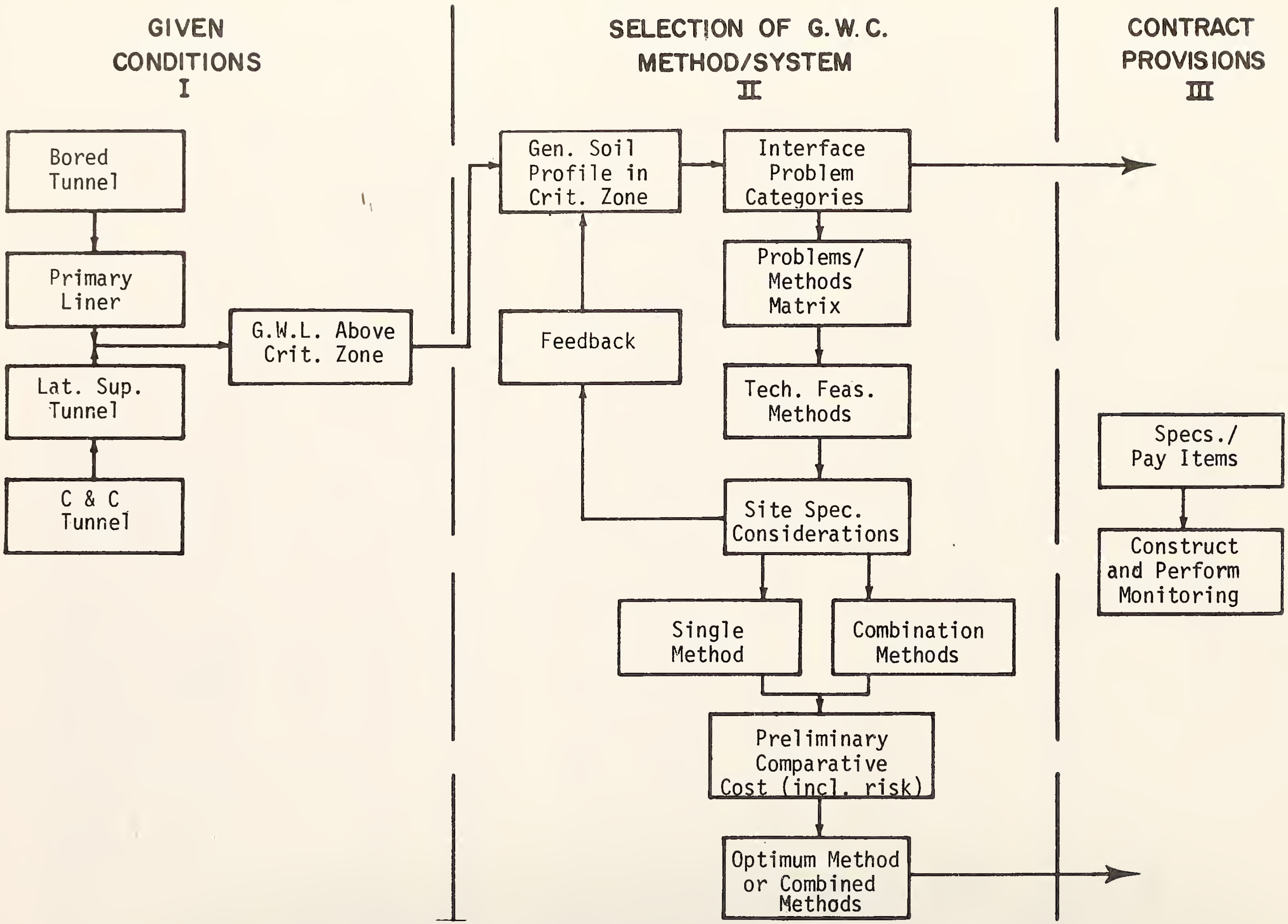
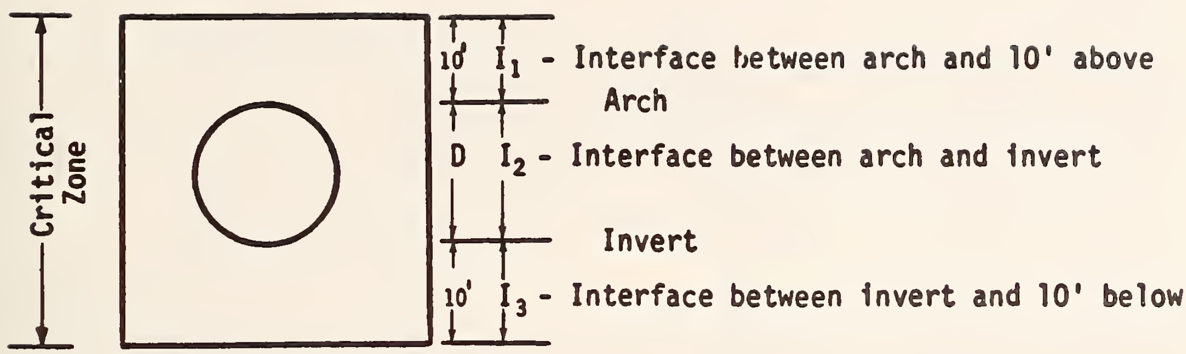
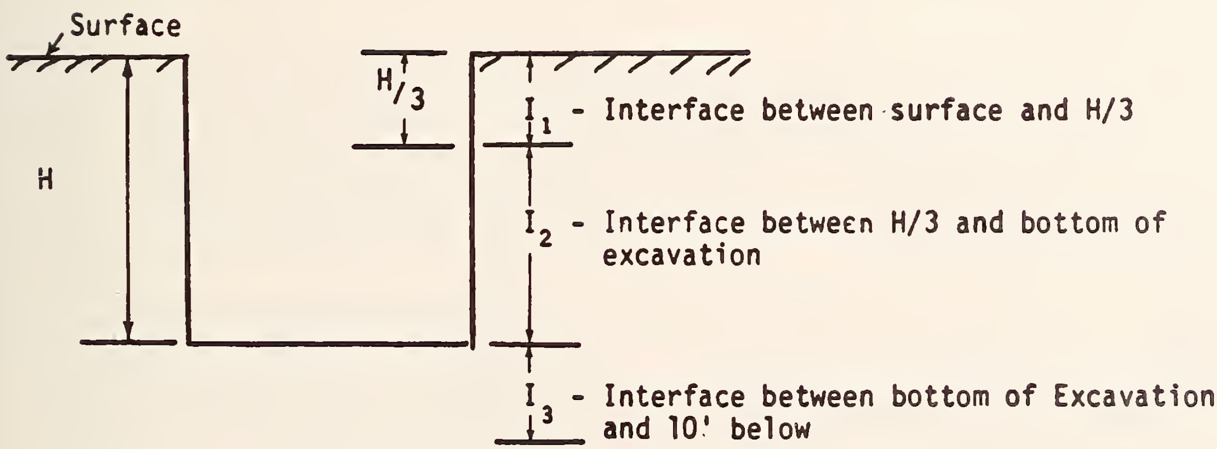


FIGURE 1 - FLOW CHART FOR USE OF SELECTION MATRICES





NOTE: 10' = 3.05m

Figure 2 - Legend Applicable to Method Selection Matrices

Method Ratings

1 - Very low

2 - Low

3 - Medium

4 - High

$\boxed{*}$ - Groundwater control method typically not considered applicable but may have some merit under unique geo-hydro conditions.

\- Groundwater control method usually not applicable.

\- No groundwater control required in soil type(s).

Notes

- Ratings are based on the technical feasibility of a single groundwater control method being successful for groundwater control in the tunnel.

- The matrix reflects the use of wellpoints, ejectors, wells, and grouting as applied externally to bored tunnels. The use of these methods internally is considered to be site specific and not typically used as a groundwater control.

- Cutoff methods, slurry walls, steel sheeting, frozen earth walls, etc., are considered to be total water cutoffs and are technically rated as 4 when terminated at or into an impermeable layer. Freezing and cutoffs are also not included because of insensitivity to soil conditions. When a cutoff is not terminated at or into an impermeable layer the matrix is used to determine a conjunctive method thereby forming a combined groundwater control method.

- For mixed face conditions combine appropriate two-layer charts to determine a rating.

- Minimal sumping may be required with any method chosen.

Assumed Soil Permeabilities

K_1 - Medium-coarse sand	$> 2 \times 10^{-2}$ cm/sec
K_2 - Fine-Medium sand	3×10^{-3} to 2×10^{-2} cm/sec
K_3 - Silt-fine sand	1×10^{-4} to 3×10^{-3} cm/sec
K_4 - Clay	0 to 1×10^{-4} cm/sec
K_5 - Fractured rock	3×10^{-3} to $> 2 \times 10^{-2}$ cm/sec
K_6 - Stratified sand and gravel	3×10^{-3} to $> 2 \times 10^{-2}$ cm/sec

Figure 3 - Notes Applicable to Method Selection Matrices

UNIFORM PROFILE

CRITICAL ZONE IN		WELLPOINTS	EJECTORS	WELLS	SUMPING	GROUTING	COMPRESSED AIR	SLURRY/EPB SHIELDS
MED. SAND OR LGR	K_1	4	2	4	1	3		
FINE SAND	K_2	4	3	3	*	1		
V.F. SAND & SILT	K_3	4	4	1	*	*		
CLAY	K_4							
FRACTURED ROCK	K_5			4	4	4		

2 LAYER PROFILE: SAND/___

SAND/___ SOIL TYPE IN CRITICAL ZONE	INTERFACE LOCATION	WELLPOINTS	EJECTORS	WELLS	SUMPING	GROUTING	COMPRESSED AIR	SLURRY/EPB SHIELDS
$\frac{K_6}{K_3}$ $\frac{\text{SAND}}{\text{SILT}}$	I_1	4	3	1	*	*		
	I_2	4	3	2	*	*		
	I_3	4	3	3	1	1		
$\frac{K_6}{K_4}$ $\frac{\text{SAND}}{\text{CLAY}}$	I_1	3	3	1	2	3		
	I_2	3	3	2	2	2		
	I_3	3	3	2	2	1		
$\frac{K_6}{K_5}$ $\frac{\text{SAND}}{\text{FR}}$	I_1	3/2	2/1	2/4	2/	3/2		
	I_2	4/2	3/1	2/4	2/	2/1		
	I_3	3/1	3/1	3/4	2/	2/1		

*/Δ   * $K_5 < K_6$ $\left(\frac{\text{COARSE}}{\text{FINE}}\right)$

Δ $K_6 < K_5$ $\left(\frac{\text{FINE}}{\text{COARSE}}\right)$

2 LAYER PROFILE: SILT/___

SILT/___ SOIL TYPE IN CRITICAL ZONE	INTERFACE LOCATION	WELLPOINTS	EJECTORS	WELLS	SUMPING	GROUTING	COMPRESSED AIR	SLURRY/EPB SHIELDS
$\frac{K_3}{K_6}$ $\frac{\text{SILT}}{\text{SAND}}$	I_1	3	2	4	*	2		
	I_2	3	3	4	*	2		
	I_3	3	3	4	*			
$\frac{K_3}{K_4}$ $\frac{\text{SILT}}{\text{CLAY}}$	I_1	3	4					
	I_2	3	4		*			
	I_3	3	4		*			
$\frac{K_3}{K_5}$ $\frac{\text{SILT}}{\text{FR}}$	I_1			4	3	2		
	I_2			4	2	2		
	I_3	1	1	4	*			

2 LAYER PROFILE: CLAY/___

CLAY/___ SOIL TYPE IN CRITICAL ZONE	INTERFACE LOCATION	WELLPOINTS	EJECTORS	WELLS	SUMPING	GROUTING	COMPRESSED AIR	SLURRY/EPB SHIELDS
$\frac{K_4}{K_6}$ $\frac{\text{CLAY}}{\text{SAND}}$	I_1	4	2	4		2		
	I_2	4	2	4		3		
	I_3	3	2	4		3		
$\frac{K_4}{K_3}$ $\frac{\text{CLAY}}{\text{SILT}}$	I_1	4	4			*		
	I_2	4	4			*		
	I_3	4	4					
$\frac{K_4}{K_5}$ $\frac{\text{CLAY}}{\text{FR}}$	I_1	2	2	4		4		
	I_2	2	2	4		4		
	I_3	2	2	4		4		

Figure 4 - Groundwater Control Method Selection Matrices for Cut-and-Cover Tunnels

UNIFORM PROFILE

CRITICAL ZONE IN	WELLPOINTS	EJECTORS	WELLS	SUMPING	GROUTING	COMPRESSED AIR	SLURRY/EPB SHIELDS
MED. SAND OR LGR. K_1		2	4	1	3	2	2
FINE SAND K_2		3	3	*	1	3	3
V.F. SAND & SILT K_3		4	1	*	*	4	4
CLAY K_4							
FRACTURED ROCK K_5		2	4	4	4		

2 LAYER PROFILE: SAND/__

SAND/__ SOIL TYPE IN CRITICAL ZONE	INTERFACE LOCATION	WELLPOINTS	EJECTORS	WELLS	SUMPING	GROUTING	COMPRESSED AIR	SLURRY/EPB SHIELDS
$\frac{K_6}{K_3}$ $\frac{\text{SAND}}{\text{SILT}}$	I_1		3	1	*	*	4	4
	I_2		3	2	*	*	4	3
	I_3		3	3	1	1	3	2
$\frac{K_6}{K_4}$ $\frac{\text{SAND}}{\text{CLAY}}$	I_1							
	I_2		3	1	*	2	4	3
	I_3		3	2	1	1	3	2
$\frac{K_6}{K_5}$ $\frac{\text{SAND}}{\text{FR}}$	I_1		2/1	2/4	2/*	3/2	*/*	
	I_2		3/1	2/4	2/*	2/1	*/*	
	I_3		3/1	3/4	2/*	2/1	*/*	

*/Δ

* $K_5 < K_6$ $\left(\frac{\text{COARSE}}{\text{FINE}}\right)$

Δ $K_6 < K_5$ $\left(\frac{\text{FINE}}{\text{COARSE}}\right)$

2 LAYER PROFILE: SILT/__

SILT/__ SOIL TYPE IN CRITICAL ZONE	INTERFACE LOCATION	WELLPOINTS	EJECTORS	WELLS	SUMPING	GROUTING	COMPRESSED AIR	SLURRY/EPB SHIELDS
$\frac{K_3}{K_6}$ $\frac{\text{SILT}}{\text{SAND}}$	I_1		2	4	*	2	4	4
	I_2		3	4	*	2	4	4
	I_3		3	4		*	4	4
$\frac{K_3}{K_4}$ $\frac{\text{SILT}}{\text{CLAY}}$	I_1						1	1
	I_2		4		*	*	4	4
	I_3		4		*	*	4	4
$\frac{K_3}{K_5}$ $\frac{\text{SILT}}{\text{FR}}$	I_1			4	3	4		
	I_2		2	4	*	3	4	
	I_3		2	4	*	*	4	

2 LAYER PROFILE: CLAY/__

CLAY/__ SOIL TYPE IN CRITICAL ZONE	INTERFACE LOCATION	WELLPOINTS	EJECTORS	WELLS	SUMPING	GROUTING	COMPRESSED AIR	SLURRY/EPB SHIELDS
$\frac{K_4}{K_6}$ $\frac{\text{CLAY}}{\text{SAND}}$	I_1		2	4		2	4	4
	I_2		2	4		3	4	4
	I_3		3	4		3	2	2
$\frac{K_4}{K_3}$ $\frac{\text{CLAY}}{\text{SILT}}$	I_1		4			*	4	4
	I_2		4			*	4	4
	I_3		4				1	1
$\frac{K_4}{K_5}$ $\frac{\text{CLAY}}{\text{FR}}$	I_1		2	4	4	4	4	
	I_2		2	4	4	4	4	
	I_3		2	4	1	4	2	

Figure 5 - Groundwater Control Method Selection Matrices for Bored Tunnels

PRESSURE RELIEF IN		WELLPOINTS	EJECTORS	WELLS	SUMPING	GROUTING	COMPRESSED AIR	SLURRY/EPB SHIELDS
SAND	K_6	3	2	4				
SILT	K_3	4	4	*				
FR	K_5	2	2	4		3		

PRESSURE RELIEF IN		WELLPOINTS	EJECTORS	WELLS	SUMPING	GROUTING	COMPRESSED AIR	SLURRY/EPB SHIELDS
SAND	K_6		2	4			*	*
SILT	K_3		4	1			*	*
FR	K_5		2	4	1	3	*	*

Figure 6 - Method Selection Matrices for Pressure Relief

Such a systems approach is not applicable for selection of methods applicable to completed tunnels and, therefore, key elements of each method of water exclusion from completed tunnels is presented in the descriptive format of Section 3.30.

3.20 DURING CONSTRUCTION

As shown in Figure 1, the selection process can be divided into 3 phases:

Phase I	Definition of given project conditions
Phase II	Selection of optimum method
Phase III	Development of contract provisions and construction monitoring

Phase I is merely project definition, i.e. is it a bored or cut-and-cover tunnel? What lateral support system for the cut-and-cover situation or what primary liner for a bored tunnel is contemplated? Once this basic information is developed, the process proceeds into Phase II, which starts with definition of the soil profile in the critical zone. Once this is done, special problems at geologic interfaces are addressed as defined in the matrices, whether it be a stratified or uniform profile. The various interface situations are described in Figures 2 and 3 are used in the matrices in Figures 4 through 6.

Once the interface problems are defined, the selection matrix is used as described in Figure 1 and technically feasible methods are selected. These are only technically feasible methods. No cost estimates have been prepared. Depending on assumptions made with regard to the soil profile, one may wish to go back through a feed-back loop, varying the assumptions of soil profile and interface conditions to consider possible alternative methods.

After technically feasible methods are selected, site specific considerations must be input into the selection process and from this will come single or multiple feasible methods.

Preliminary cost estimates are then performed, which results in selection of an optimum method or combination of methods. A meaningful cost estimate includes not only the cost of the dewatering system, but must also consider the impact of the system on the entire project.

Once optimum methods have been selected, the process moves into Phase III, which is development of contract provisions and construction performance monitoring.

Cost estimates are not presented herein because the cheapest groundwater control method is not always the most cost effective when other factors are considered. Consideration must be given

to the effect of a given method on other construction processes and on the environment. For instance, grouting may be a more expensive option initially than dewatering, but the increased face stability resulting from grouting could lead to a cheaper total tunnel cost due to increased productivity. Similarly, the installation of a pre-drainage system may be inexpensive and reliable, yet may cause settlement in overlying compressible soils which could cause building distress in urban areas. As a result, each project must be considered individually and with a full knowledge of the implications of the dewatering method chosen. For a more through discussion of cost implications, refer to Section 5.30 of Volume 1.

As an aid to the selection of technically feasible methods, a series of selection matrices have been developed which account for general geologic conditions, (i.e., permeability and stratification), and construction procedure, (i.e., bored or cut-and-cover methods). These matrices are presented in Figures 3 through 6.

Selection is based on subjective rating system varing from 1 to 4 as follows:

1=very low
2=low
3=medium
4=very high

These ratings have been superimposed on the system of ten matrices, i.e., 4 for cut-and-cover tunnels, 4 for bored tunnels, and 2 for pressure relief considerations. These ratings have been determined, as previously noted, based upon technical criteria. For example, predrainage methods are feasible in nearly all granular soils with the highest rating applying to the drainage method which is most effective in that stratum. Deep wells are rated highly for sands, whereas ejectors are more effective in fine sands and silts. Wellpoints are feasible in coarse sands as well as silts, but are limited in depth and so may be useless in any deep (bored tunnel) scheme. None of the ratings for predrainage methods take into account factors such as adjacent settlement, recharge requirements, chemical effects or water disposal. These are all site dependent variables which cannot be quantified here.

Cutoff systems such as slurry walls and trenches, steel sheeting and freezing are all considered "perfect" dewaterng techniques when properly installed and thus are all rated "4." These systems are, therefore, not included in the matrices. Table 7 is a summary of groundwater control methods during construction. A more thorough description of each method is included in Section 4.00 herein and in a corresponding section of Volume 1.

TABLE 7 SUMMARY OF GROUNDWATER CONTROL METHODS DURING CONSTRUCTION (Sheet 1 of 3)

METHOD	APPLICATION IN TUNNEL DEWATERING	ADVANTAGES	LIMITATIONS
Predrainage			
Deep wells	- Sand and gravel	- Wide spacing - Large volumes pumped	- May result in pumping a greater quantity than necessary
Well points	- Sand and gravel	- Spot control possible - Minimize quantities pumped	- Lift limited to 15 - 20 feet
Ejectors	- Sand, silty sand		- Inefficient - Susceptible to encrustation
Vacuum wells	- Silty sand, sandy silt	- Increase yield when gravity drainage is slow	- Difficult to maintain - Complex
Pumping within Tunnel	- In stable water-bearing strata	- Inexpensive - Simple	- Can lead to unstable soils due to seepage pressures.
Cutoffs			
Steel sheeting	- Nearly any pervious or semi-pervious soil through which it can be driven	- Materials reusable - Easily installed - Effective in pervious soils	- Sheeting must be driven intact - Cobbles or obstructions may damage sheeting - Seepage thru interlocks can limit effectiveness in semi-pervious soil

TABLE 7 SUMMARY OF GROUNDWATER CONTROL METHODS DURING CONSTRUCTION (Sheet 2 of 3)

METHOD	APPLICATION IN TUNNEL DEWATERING	ADVANTAGES	LIMITATIONS
Slurry walls	- Any soil which will contain slurry, i.e. gravelly sand to clay	- Wall can be used as part of structure - Leaks can be repaired during excavation - Many variations in basic technique	- Boulders difficult to excavate - Messy operation
Slurry trench cutoffs	- Any soil	- Inexpensive - Able to withstand lateral movements	- Proper construction required to avoid non-homogeneous backfill
Thin wall cutoffs	- Any soil through which probe can be driven	- Economical	- Must penetrate impervious stratum - Thin cutoff
Recharge	- Sand and gravel	- Minimizes settlement outside excavation	- Encrustation of pipes - Entrained air in pipes - Must be monitored
Grout	- Gravel to sandy silt	- Very effective in eliminating flow into tunnel - Stabilize soils unstable due to seepage pressures	- Difficult to monitor - Environmental concerns with chemical grouts

TABLE 7 SUMMARY OF GROUNDWATER CONTROL METHODS DURING CONSTRUCTION (Sheet 3 of 3)

METHOD	APPLICATION IN TUNNEL DEWATERING	ADVANTAGES	LIMITATIONS
Compressed air	- Pervious sands, silty sand - Sandy silt	- Very effective silt and fine sands - Limits settlement outside tunnel	- Large air losses in very pervious soils (gravels) - Strict regulation of working environment - Limited work shifts in pressures over 12 psi - Possible "blowout"
Slurry Shields	- Silt and silty sand - Granular soils with no large cobbles	- Stable in water bearing sands - Minimizes surface settlement - Minimal tunnel hazards - Accurate tunnel alignment	- Problems in mixed face conditions - Large cobbles a problem - Cohesive soils a problem
Electro-osmosis	- Very fine grained silts, sandy or clayey silts	- Reverses seepage in open cuts - Stabilizes runny or sloughing soils - Consolidates and increases strength near cut - Useable where gravity or vacuum drainage is too slow	- Expensive - Requires continuous power - Difficult to assess power requirements

3.30 SELECTION OF GROUNDWATER CONTROL METHODS IN COMPLETED TUNNELS

The main factors which control the choice of a permanent groundwater control method are the type of tunnel and the method of construction. Table 8 is a summary of available methods of permanent groundwater control and the following paragraphs concisely describe the advantages and limitations of each of these methods. Volume 2 contains additional descriptive material.

3.31 Cast-in-Place Concrete

Cast-in-place concrete liners can be used in nearly all tunnel types as either a primary or secondary method of groundwater control. As a primary means of groundwater exclusion they are most useful in cut-and-cover tunnels due to the room available for formwork, concrete mixing and placement. In soft ground tunnels the cast-in-place method is less applicable due to the problems associated with concrete placement in a confined, wet environment. Where the concrete can be placed, it has the advantages of flexibility, i.e., the mix consistency, pour length, set time and joint frequency can be modified to deal with special construction conditions. Where the quality of the placed concrete is doubtful, a secondary liner or membrane may be necessary to insure watertightness.

Cast-in-place concrete may also be selected as a secondary method behind pre-cast concrete or segmented steel liners in bored tunnels. Concrete is used as secondary liner material for steel shell sunken tube tunnels, although its effectiveness in increasing watertightness is minimal. In either case, the concrete provides an additional barrier to seepage and aids the structural integrity of the tunnel.

3.32 Waterproofing Membranes

Membranes are applicable in nearly all types of tunnels for excluding groundwater during the life of the structure. A variety of membranes are currently available, including built-up hot applied bituminous materials, pre-formed multilayered boards, cold applied materials, plastic or synthetic rubber sheets, bentonite panels or sprays, cementitious coating and thin metal membranes.

Waterproofing membranes are often selected for cut-and-cover tunnel sections and sunken tube tunnels because of the ease of application. The membranes can be applied to the concrete surface in cut-and-cover sections with little problem and with a reasonable amount of flexibility. The membrane can be chosen

TABLE 8. SUMMARY OF METHODS TO EXCLUDE WATER FROM COMPLETED TUNNELS

METHOD	ADVANTAGES	DISADVANTAGES	COMMENTS
Cast-in-Place Concrete Linings	Resistant to environmental deterioration Joint spacing can be easily adjusted Defects can usually be repaired Flexibility to deal with special situations in the field: - regulated set times - expansive agents - mix strengthening by additives - reinforcement with fibers	Difficult to place in confined or wet environment Good concrete placement procedures required to: - minimize shrinkage - minimize segregation - insure impermeability Must resist earth pressures as well as hydrostatic pressures Proper concrete curing is essential	More efficient and easier to control in cut and cover applications
Waterproofing Membranes General:	Most easily applied in cut and cover jobs	Can be punctured during installation or backfilling	Often used where primary and secondary liners are installed Should be flexible to move with adjoining structures without leaking
- Bituminous Materials			
Brick in asphalt mastic	Considerable previous experience	Expensive	Infrequently used at present
Built up membrane	Long term stability	Difficult to apply on vertical surfaces Susceptible to damage from downdrag forces if adjacent soil settles	Particularly effective on exterior of cut and cover tunnels
Preformed multi-layered board	Quick installation process Variable number of plies possible	Concrete surfaces must be clean and intact	
Cold applied bituminous material	Easily applied by brush or spray	Limited effectiveness	Considered a "damp proofing" method

TABLE 8. SUMMARY OF METHODS TO EXCLUDE WATER FROM COMPLETED TUNNELS (continued) 2/6

METHOD	ADVANTAGES	DISADVANTAGES	COMMENTS
- Plastic/Synthetic Rubber Sheeting			
General	Resistant to chemicals, aging puncture Lightweight Flexible, with good elastic properties Electrical resistance Self-healing to some degree		Often single layer systems
Polyethylene sheets	Nearly any size or shape	Sheets may be damaged by adjacent construction activity	
Polyvinyl Chloride Sheeting			Similar to polyethylene
Butyl Rubber Sheeting	Great flexibility Does not stiffen with age Self-healing properties Abrasion resistant Sheets easily spliced Most impervious of synthetics		Most frequently used synthetic in U.S.
Hypalon Sheeting	Large elongation before rupture Can withstand large differential movement	Expensive	
Neoprene Sheeting	Properties can be altered to suit the job		Properties vary with additives
- Cold, liquid applied waterproofing	Usually more economical than sheeting	Difficult to apply uniformly and adequately	Typically reserved for remedial waterproofing
	Can typically be sprayed, rolled or troweled into place	Mutiple coating may be necessary	One or two component systems
- Bentonite panels/sprays	Expands when wet to fill voids/crack	Susceptible to degradation by chemicals in groundwater	
Panels	Effective at any temperature	Must be installed on smooth surface and protected from damage and moisture	
Spray	May be applied to irregular surfaces		

TABLE 8. SUMMARY OF METHODS TO EXCLUDE WATER FROM COMPLETED TUNNELS (continued) 3/6

METHOD	ADVANTAGES	DISADVANTAGES	COMMENTS
- Cementitious coatings	Can be applied directly on wet surfaces	Coatings must be moist cured Expensive	
	Leaks can be easily repaired	Requires skilled hand labor	
- Aluminum sheeting	Relatively inexpensive materials	Susceptible to degradation by certain chemicals	
<u>Segmented Tunnel Linings</u>			
General	Typically combined support and waterproofing system in one	Results in more numerous sources for leaks at joints	
- Cast iron segmented linings	Highly resistant to chemical and electrolytic corrosion	Limited availability in U.S.	Lead caulking between segments, provides waterproofing
- Fabricated steel segments	Effective waterproofing, commonly used in U.S.	Subject to corrosion and must be protected	Lead caulking used between installed segments can be used with secondary
- Precast concrete segments	Potential for savings over other segmented linings	Possible sources of leaks in poor or damaged concrete	Adhesive sealers useable for low water pressures
	Flexible with respect to segment configurations	Heavier segments are difficult to handle	Caulking or gasket seals for higher pressures
	A variety of seals available to waterproof between segments	Segment are brittle and easy to damage during transport or installation	
- Segmented linings or rock tunnels	Rapid progress using pre-cast segments with tunnel boring machines	See notes above Limited flexibility to deal with unanticipated conditions	
<u>Chemical Grouting</u>			
General	Temporary as well as permanent waterproof	Requires extensive field investigation	
	Can strengthen as well as impermeabilize soil surrounding tunnel	Requires expertise to choose proper grout type, set time, infection pressure	Frequently used in Europe as a primary treatment for waterproofing
	Does not require enlarging of tunnel to accommodate waterproofing material	Difficult to monitor success of grouting process	

TABLE 8. SUMMARY OF METHODS TO EXCLUDE WATER FROM COMPLETED TUNNELS (continued) 4/6

METHOD	ADVANTAGES	DISADVANTAGES	COMMENTS
- Silicate-based grouts	Wide range of viscosity, setting times and strengths Weak solution can still penetrate fine sands Least expensive of chemical grouts	Possible syneresis	One-shot injection process most common in U.S.
- Plant-product grouts			
- Lignin grout	Wide range of setting times Relatively inexpensive Non-petroleum based Low viscosity	Reactant is toxic Imparts little strength to soils	
- Colophane	Non petroleum based	Only for waterproofing	
- Forfural	Non petroleum based	Little experience	
- Acrylamides	Well controlled set time Low viscosity	Very expensive Possible toxic effects	No longer in general use in U.S.
- Resins			
- Phenoplasts	Low viscosity; non petroleum based	Individual components are toxic	Used for strengthening as well as waterproofing
- Aminoplasts	Relatively low cost Non petroleum based	Require an acid medium to gel	
- Polyurethane foams	Good waterproofers	Two-shot injection process Toxic ingredients before setting Expensive	
- Emulsions	Non toxic Good long term stability Useful in fine and silty fine sands	Grout coagulation difficult to control	Does not strengthen soil
- Bituminous asphalts	Effective in controlling flowing	Must be heated before injection	Seldom used, little experience
- Vulcanizable oils		Expensive High initial viscosity Long set time	Seldom used, little experience
- Reactants	React directly with groundwater	Limited application	

TABLE 8. SUMMARY OF METHODS TO EXCLUDE WATER FROM COMPLETED TUNNELS (continued) 5/6

METHOD	ADVANTAGES	DISADVANTAGES	COMMENTS
Consolidation Grouting (Rock)			
General	Inexpensive	Possible segregation	Typically done using particulate cement grouts or cement-clay grouts
	Widespread experience	Admixtures needed to cope with groundwater flows	
	A variety of materials and additives available to meet job needs		
Annular Space and Contact Grouting			
- Annular space grouting	May reduce need for extensive grouting or waterproofing	Not always a reliable method for waterproofing completely	Grout pumped into voids of gravel packed into annular tunnel space
- Contact grouting	May reduce need for extensive grouting or waterproofing		Backfilling of voids between primary and secondary liners with grout
Diaphragm Walls			
General	May be incorporated into final structure	Must penetrate impervious stratum	Cut and cover project
	Constructed in any soil which can be held open by slurry	Joints require sealing or water-stops	Temporary or permanent water barrier
			May be structural as well as water barrier
- Cast-in-place concrete	Waterproof as well as structural	Expensive if not used as permanent wall	Most common type
- Cast-in-place soldier piles and tremie concrete	Waterproof as well as structural	Slurry or concrete may become contaminated with loose soil along piles Leakage at concrete/steel interface	
- Precast concrete	Good quality concrete Good waterproofing abilities Slurry behind panels increases waterproofing ability	Wall depths limited May be expensive Less flexibility to deal with unanticipated conditions	Limited use outside of Europe

TABLE 8. SUMMARY OF METHODS TO EXCLUDE WATER FROM COMPLETED TUNNELS (continued) 6/6

METHOD	ADVANTAGES	DISADVANTAGES	COMMENTS
Permanent Ground-Water Lowering	Eliminates need for extensive waterproofing	Usually impractical Adverse effects on adjacent structure Requires long-term maintenance	Never been used to date
Sunken Tube Tunnels			
General	Very long tunnel segments (few joints)	Very strict waterproofing necessary Joints are sealed underwater	Used under bodies of water (channels, harbors, etc.)
- Steel shells	Steel provides primary waterproofing membrane	The steel must be protected from corrosion, etc.	Typically welded joints; secondary concrete liner may be used
- Reinforced concrete tubes	Joint flexibility possible	Requires dense, high quality concrete	Usually covered with steel or asphaltic membrane Joints sealed with gaskets

based upon construction or site variables such as concrete condition, extreme weather or water conditions, frequency of joints in the concrete, possible shrinkage conditions, possibility of puncture, magnitude of expected movements and economics.

Waterproofing membranes may be used as secondary liners in bored tunnels but usually require a smooth outer liner upon which the membrane can be placed.

3.33 Segmented Tunnel Linings

Segmented linings are used in bored soil or rock tunnels as combined structural and waterproofing systems. The segments can be fabricated of cast iron, structural steel or pre-cast concrete.

Precast concrete segments are flexible with respect to segment size and configurations but are heavier and more difficult to handle than steel sections.

Segmented tunnel liners are used frequently in soft ground tunnel construction and to a lesser degree in rock tunnels. A disadvantage to the method is the frequency of joints associated with the large number of segments required. However, leaks are easily repaired by re-caulking the joints or grouting behind the liner.

3.34 Grouting

Grouting adjacent to the excavation is a dewatering technique applicable in both cut-and-cover and bored tunnels. The type of grout used depends upon a number of variables discussed in the companion volumes of this report. The available materials most commonly used are cement, cement-bentonite, silicates, lignin-based, acrylamides, resins and emulsions. An advantage of grouting is that it can be used as a dewatering method both durng constuction and over the life of the tunnel. Its use is limited in fine grained cohesionless or stratified soils where the viscosity of most grouts is too high to effectively permeate the stratum and form the impervious barrier. Good knowledge of the soil conditions is required prior to selection of a grouting system.

Consolidation grouting of rock is a widely used and inexpensive technique for limiting water flows through rock fractures and seams. This method is commonly used in conjunction with other techniques to form an impervious barrier around the tunnel, and as such may be termed a secondary treatment. The grouting is carried out using various mixes of particulate grouts (cement, clay, sand) injected from the surface or directly into rock seams as the tunneling proceeds.

Contact grouting and annular space grouting are simply the filling of voids which may exist behind the tunnel lining with a particulate mix of grout or wet concrete. The grout is pumped into predetermined hole patterns to insure adequate backfilling and minimize segregation or trapped pockets of air. This technique is usually a secondary defense against water intrusion and ground movement above the structural liner.

3.35 Diaphragm Cutoff Walls

Diaphragm walls are used exclusively in cut and cover tunnel sections because they can be incorporated into the final structure. The type of wall may be cast-in-place, solder pile and tremie concrete, or pre-cast concrete. Leakage can occur through the many joints associated with the wall construction, but is usually easily remedied if it occurs above excavation levels. The concrete cutoffs must penetrate an impervious stratum to be effective and the construction procedure must be good to insure high quality concrete.

3.36 Permanent Groundwater Lowering by Drainage

This method has never been attempted on a transit tunnel but is technically feasible. The main objection to its use is the possible adverse affect on adjacent structures in urban areas.

3.37 Sunken Tube Tunnels

Such tunnels are used primarily in tunnel construction beneath large bodies of water such as harbors, channels, or rivers. The waterproofing capabilities of these sections are high because they are fabricated under controlled conditions and have joints at spacings of 100 feet (30.5m) or greater.

4.00 DESIGN AND CONSTRUCTION OF GROUNDWATER CONTROL METHODS FOR URBAN TUNNELING

The design and construction of groundwater control methods for use during construction of tunnels in urban areas are so closely interrelated that it is often difficult to determine where design ends and construction begins. This is particularly true of dewatering and recharge methods, grouting and freezing. Cut-offs, compressed air and specialty shields, i.e., slurry or earth pressure balance shields, are less subject to modification during construction than the previously listed methods. That is not to say that there are no construction difficulties with these methods, but the basic concept or layout is less likely to change.

4.10 DEWATERING SYSTEMS

4.11 General Design Considerations

The following factors should be evaluated for design of predrainage systems.

1.) Permeability - Soil permeability can be estimated based on grain size analyses, laboratory permeability tests and borehole permeability tests. Rock mass permeability is usually estimated based on borehole packer tests and pumping tests. Grain size tests and borehole pumping tests are the most reliable methods for evaluation of soil permeability and are described below.

Grain Size Analyses

- Grain size analysis using Hazen's relationship $K = 100D^2_{10}$ cm/sec, is useful for uniform sands. The relationship is not as well suited to well graded granular soils and in such situations qualitative estimates based on general soil classifications are often used. Some typical values are listed below.

SOIL	Permeability (cm/sec.)
Coarse Gravel	11×10^{-2}
Sandy Gravel	1.6×10^{-2}
Fine Gravel	0.7×10^{-2}
Silty Gravel	4.6×10^{-4}
Coarse Sand	1.1×10^{-4}
Medium Sand	2.9×10^{-3}
Fine Sand	1.0×10^{-3}
Silt	1.5×10^{-5}

Pumping Tests

- Pumping tests are probably the most reliable way to estimate aquifer permeability. Two simple closed form solutions for steady state pumping are;

Unconfined Aquifers

$$K = \frac{458\ Q\ \ln r_2/r_1}{(h_2^2 - h_1^2)} \text{ in gpd/ft}^2$$

Confined aquifers

$$K = \frac{229\ Q\ \ln r_2/r_1}{m\ (h_2 - h_1)} \text{ in gpd/ft}^2$$

where:

K = Coefficient of permeability in gpd/ft^2

Q = Flow in gpm

r_2 = Distance to farthest observation well in ft.

r_1 = Distance to closest observation well in ft.

h_2, h_1 = Saturated thickness in ft. at furthest and closest wells, respectively

m = Thickness of confined aquifer in ft.

These are unique mathematical solutions which must be used with considerable judgement. Somewhat more realistic methods of interpreting pumping test data are illustrated conceptually in Figure 7a. For a thorough discussion of methods such as shown in Figure 7a, the reader is referred to anyone of several tests such as Groundwater Resource Evaluation, by William Walton and published in 1970 by McGraw-Hill Publishing Co., New York.

Variable and constant head borehole permeability tests, although less accurate than pumping tests, are inexpensive indicators of permeability and are therefore frequently run. Figure 7b presents the basis for two common borehole tests.

Note: gpd/ft^2 = 4.716×10^{-7} m^3/S/m^2

= 0.040747 m^3/day/m^2

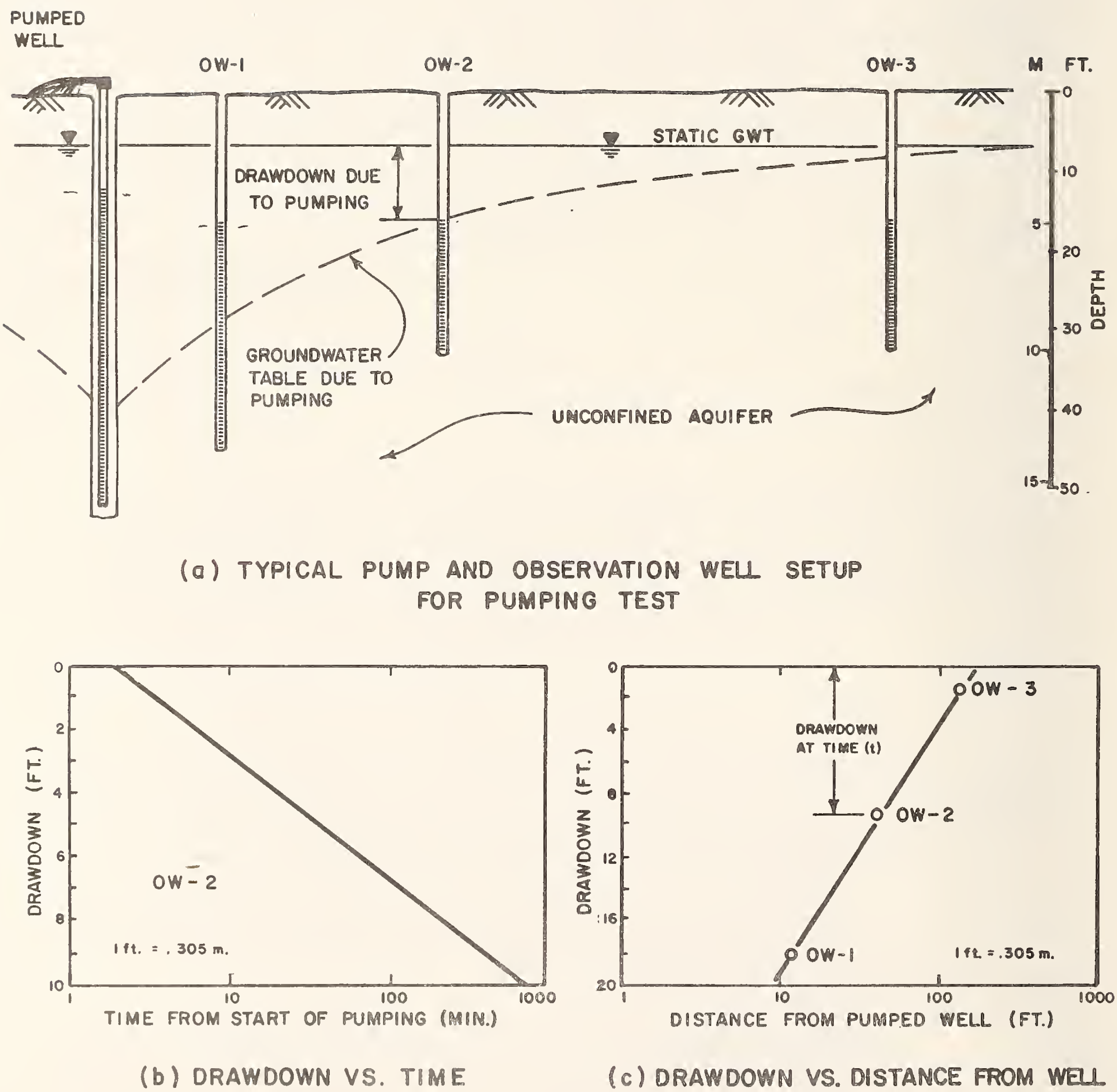
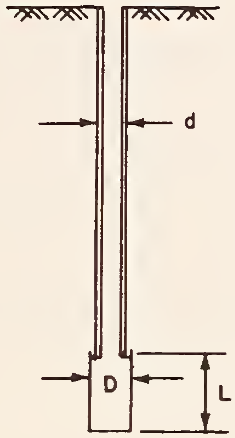
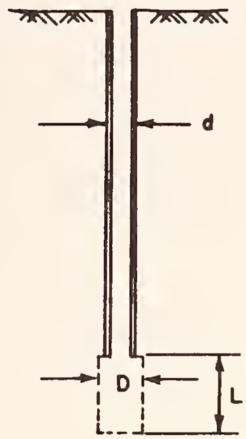


FIGURE 7a - Typical Pumping Test Array and Data

	 CASED HOLE; SOIL FLUSH WITH BOTTOM	 CASED HOLE; UNCASED OR PERFORATED EXTENSION OF LENGTH, L
CONSTANT HEAD TEST	$K_m = \dfrac{q}{2.75 D H_c}$	$K_h = \dfrac{q \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2 \pi L H_c}$
VARIABLE HEAD TEST	$K_m = \dfrac{\pi d^2}{11 D (t_2 - t_1)} \ln \dfrac{H_1}{H_2}$	$K_h = \dfrac{d^2 \ln\left(\frac{2mL}{D}\right)}{8L(t_2 - t_1)} \ln \dfrac{H_1}{H_2}$

NOTATION:

K_m = MEAN PERMEABILITY $= \sqrt{K_h \cdot K_v}$

K_h = HORIZONTAL PERMEABILITY

K_v = VERTICAL PERMEABILITY

q = FLOW OF WATER

L = LENGTH OF INTAKE ZONE

H_c = CONSTANT PIEZOMETRIC HEAD

H_1, H_2 = PIEZOMETRIC HEAD AT t_1, t_2

d = DIAMETER OF STANDPIPE OR CASING

D = DIAMETER OF INTAKE ZONE

m = $\sqrt{K_h / K_v}$, t = TIME

FIGURE 7b - Typical Field Permeability Test Methods

2.) Water Bearing Zone - The thickness of the water bearing zone is normally estimated from boring logs and piezometer measurements

3.) Storage Coefficient S - The storage coefficient is estimated from pumping tests. As a rule of thumb for a confined aquifer $S = m \times 10^{-6}$

Ranges of S values are typically 0.0001 to 0.001 for a confined aquifer and 0.01 to 0.35 for an unconfined aquifer.

4.) Radius of Influence - The radius of influence can be estimated from pumping test data.

As an estimate, $R_o = \sqrt{\frac{0.3\ Tt}{S}}$

where:

R_o = Radius of influence in feet

T = Transmissivity in gpd/ft.

t = Time of pumping in days.

5.) Estimated Pumping Quantity - The total quantity of water to be pumped, Q, during steady-state conditions is given by;

$$Q_T = \frac{K\ (H^2 - h^2)}{458\ \ln R_o/r_w} + \frac{X\ K\ (H^2 - h^2)}{1440\ R_o}$$ for an unconfined aquifer

or

$$Q_T = \frac{Km\ (H - h)}{229\ \ln R_o/r_w} + \frac{X\ Km\ (H-h)}{720\ R_o}$$ for a confined aquifer

where:

Q_T = Pumping rate; gpm

K = Coefficient of permeability, gpd/ft

H = Saturated thickness of aquifer before pumping (unconfined aquifer) or

H = Static head at bottom of aquifer before pumping (confined aquifer)

h = Saturated thickness of aquifer after pumping (unconfined aquifer) or static head at bottom of aquifer after pumping (confined aquifer)

R_o = Radius of influence in ft.

r_w = Radius of well in ft.

X = Required length of dry tunnel

m = thickness of confined aquifer, in ft.

The pumping rate for a single well can be conservatively estimated based on:

$$Q_w = .035\ l_s\ r_w\ \sqrt{K},$$

where:

Q_w = Pumping rate in gpm

l_s = Length of wetted screen after drawdown, in ft.

r_w = Well radius in inches

K = Coefficient of permeability in gpd/ft.

4.12 Design of Well Systems

A typical deep well is illustrated in Figure 8. A preliminary estimate of the number of wells can be made by:

$$Nw = \frac{Q_T}{Q_W}$$

where:

N_w = Number of wells

Q_T = Total flow to dewater the tunnel

Q_w = Flow per well

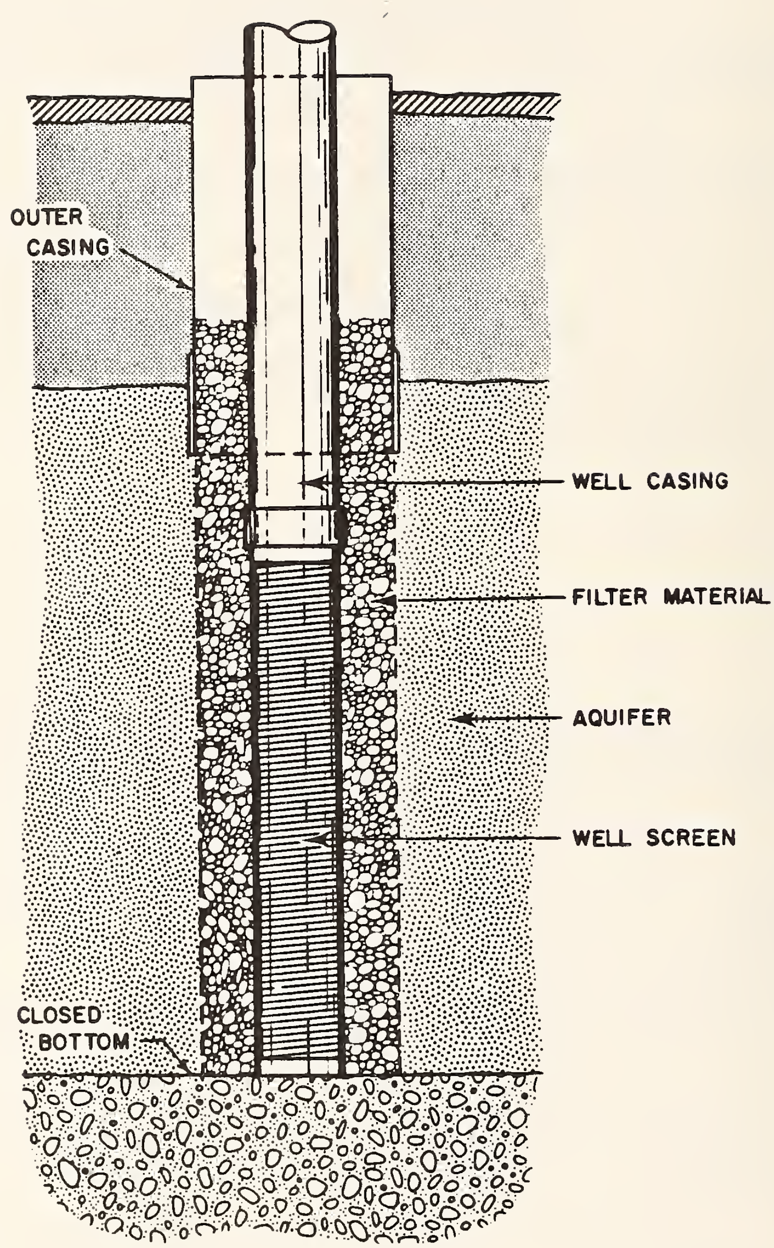


FIGURE 8 - Typical Deep Well

The preliminary system design is checked by making a cumulative drawdown analysis at points of required maxiumum drawdown. Also checks at critical well locations are made to assure adequate wetted screen length, l_s. The well and formation head losses will reduce the available l_s calculated. Individual well flows and/or well spacing and number are adjusted until the desired drawdown requirements are met. When l_s available is much greater than l_s required, it is probable that fewer wells with higher individual flows may be adequate. When l_s available is much less than l_s required, more wells with lower individual flows may be adequate. An estimate of cumulative drawdown can be made using the following:

$$(H-h)_{1\rightarrow n} = \sum_{1}^{n} \left[H - \sqrt{H^2 - \frac{Q_n\ 458 \ln R_o/r_n}{K}} \right] \text{ in ft. for unconfined aquifer}$$

$$(H-h)_{1\rightarrow n} = \sum_{1}^{n} \left[\frac{Q_n\ 229 \ln R_o/r_n}{Km} \right] \text{ in ft. for confined aquifer}$$

Filter material is selected based on compatibility with formation soil type. A compromise on filter material may be required if pumping is to be undertaken in varying soil types. It is also based on compatibility with the wellscreen. Generally, the gradation of filter material should be within the following limits:

Filter to aquifer relationship:

$$\frac{D_{15}\text{ (filter)}}{D_{85}\text{ (aquifer)}} < 4 \text{ to } 5 < \frac{D_{15}\text{ (filter)}}{D_{15}\text{ (aquifer)}}$$

Filter to well screen relationship:

$$\frac{D_{85}\text{ (filter)}}{\text{screen slot or hole}} > 1 \text{ to } 2$$

If water quality data indicate a corrosive or encrusting environment, modification in slot size or screen material may be necessary.

The collection system is sized and the discharge is located based on normal hydraulic principles or pipe capacities. The pumps are sized based on total flow, total dynamic head, net positive suction head and pump efficiency.

If pretreatment of discharge is required, the system should be designed in accordance with common sanitary engineering practices.

Power requirements and selection of backup power systems are selected after selection of pumps.

4.13 Design of Wellpoint Systems

Typical wellpoint installations are illustrated in Figure 9. The design procedure for a wellpoint system first involves distribution of the calculated total flow, Q_T, to the perimeter of the well point system, to get an estimated required flow per foot of system. The wellpoint size, and spacing between wellpoints are than chosen to be compatible with the material and flow rates anticipated.

Friction losses in wellpoints should be kept to a minimum. A perfect vacuum at sea level has the theoretical potential to lift water 34 feet (10.4m). The actual lift available below pump suction elevation at a wellpoint is determined by deducting the following head losses in feet from 34 feet (10.4m):

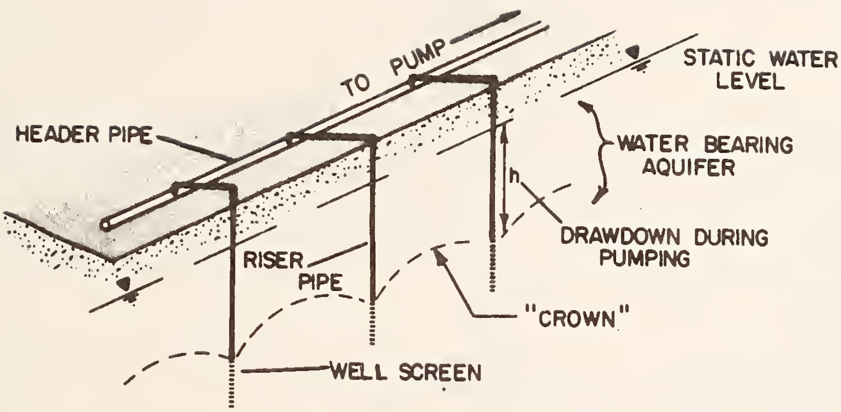
- Friction loss through screens
- Friction loss through piping
- Pump losses (efficiency)
- Reduction in barometric pressure above sea level

Typically these effects reduce the available lift at the wellpoint to about 24 feet (7.3m) at sea level. As a rule of thumb, 15-foot (4.6m) drawdown in the center of the excavation is the maximum drawdown that shuld be anticipated for a single stage wellpoint system.

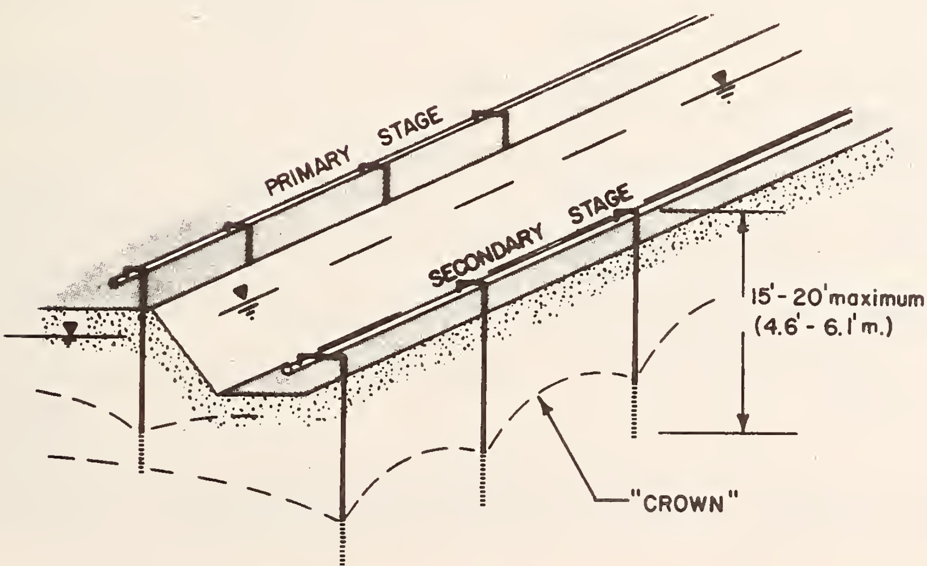
If sufficient drawdown is not produced by a single stage of wellpoints, a second stage must be provided at the lower elevation. The drawdown elevation of the first stage is then used as the static water level of the second stage. When multiple wellpoint stages are required, the headers are usually connected to a common vacuum source.

Typically commercial wellpoints are designed with screen openings that will retain a washed concrete sand. If the aquifer soil is so fine that it may clog a concrete sand filter, a specific screen opening and filter material may have to be provided in accordance with filter guidelines discussed for wells.

The size and location of discharge lines and pre-treatment facilities, if required, should be as discussed for wells.



a) SINGLE STAGE



b) TWO STAGE

FIGURE 9 - Typical Wellpoint Installations

Determine power requirements and backup power system. If a diesel powered pump is used, some flexibility in pumping rates is possible by running the pump at higher speeds. Electric pumps run only at one speed, but require less fuel.

4.14 Design of Ejector Systems

A typical ejector is illustrated in Figure 10. The design procedure for ejector systems first involves making an estimate of Q_T. Then determine the head between screen and discharge line and chose an ejector spacing. Spacing is normally between 7.5 and 20 feet (2.3m to 6.0m). In stratified soils where only a moderate amount of seepage can be tolerated, spacing closer than 7.5 feet (2.3m) may be necessary.

The number of ejectors to be used can be calculated by dividing Q_T by the yield per ejector, Q_2.

The nozzle size, supply flow Q_1, and supply pump pressure required to obtain Q_2 are obtained from ejector manufactures' rating charts. Based on the required values of Q_2 and discharge head, a nozzle size is specified with appropriate values of Q_1, and supply pressure. If Q_2 is beyond the limits of nozzle ratings, the ejector spacing must be adjusted accordingly.

The size of the supply pump is determined based on required supply pressures and total supply flow, Q_{1T}. Friction in both supply side and return side piping must be accounted for.

The supply header is sized based on a total supply flow and discharge header is sized based on total return flow, as double the normal value, to allow for flow of air and water. Since the proper operation of the ejector is sensitive to back pressure, efforts should be taken to minimize friction losses in the return and discharge lines. This can be acheived by the installation of air vents in the header and, when possible, providing multiple discharge points in the system.

Filter material and ejector screens are selected as described for design of wells.

The ejector pump is very sensitive to encrustation in the nozzle and venturi. It may therefore be necessary to design pre-treatment facilities.

Power requirements and backup power systems can be designed as described in normal design of mechancial systems.

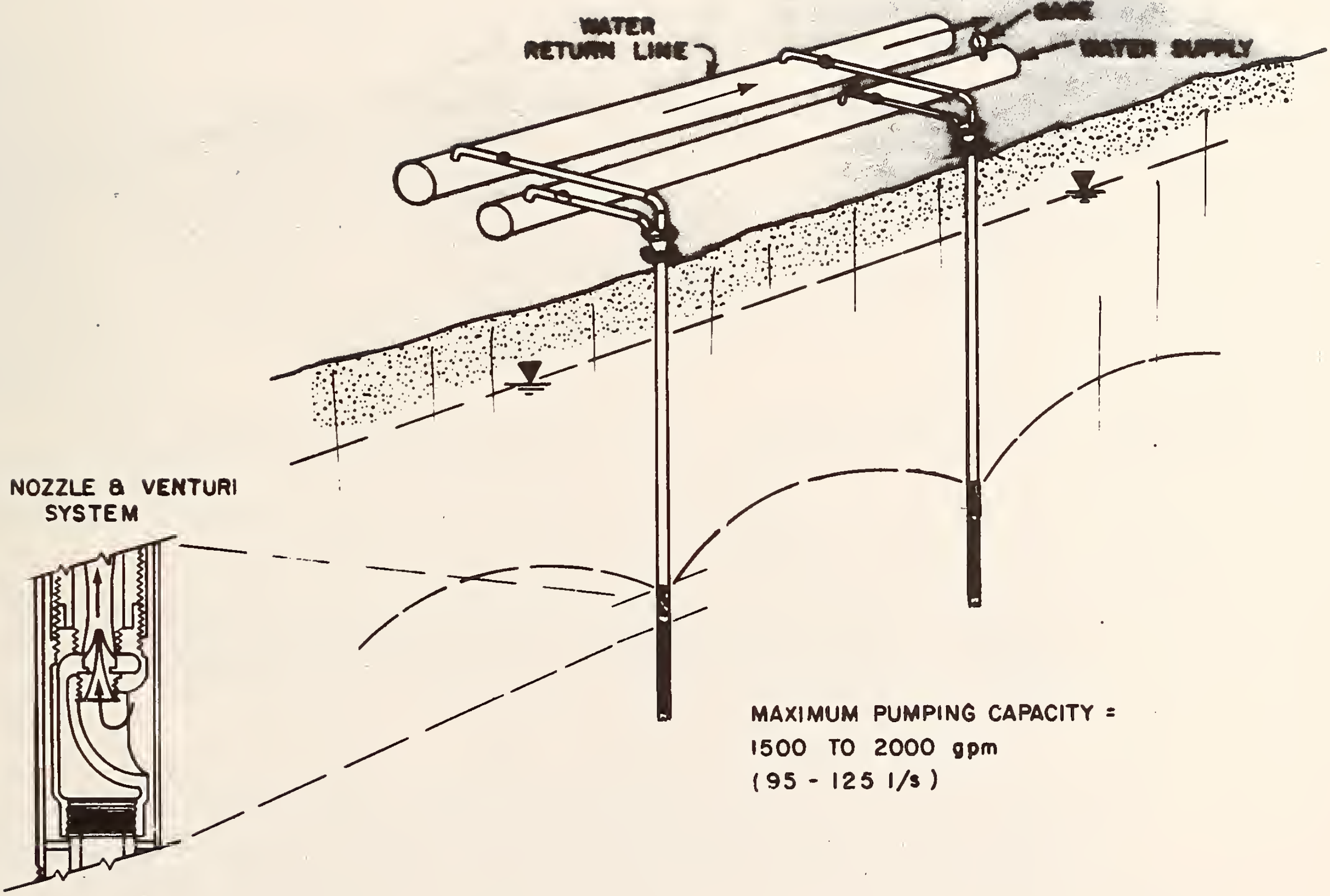


FIGURE 10 - Typical Ejector System

4.15 Construction of Dewatering Systems

Successful operation of dewatering systems requires careful attention to details of construction. Some of the more important considerations include:

1. The filter material and screen slot sizes must be compatible with soil conditions as discussed in Section 4.12.

2. The filter must be placed over the top of the screen to allow for settlement and volume loss during well development. If the well is sealed, a pipe is sometimes installed to the top of the filter for the purpose of adding additional material, if necessary.

3. The well should be thoroughly developed to minimize pumping of fines and to maximize capacity.

4. Standby equipment should be available, particularly in pressure relief situations. This consideration is less critical in unconfined aquifers which respond much more slowly to changes in pumping rates. Equipment should be available in such a case, but does not need immediate response time.

5. Areas susceptible to freezing such as low flow areas or "dead ends" on discharge headers, should be insulated.

6. Deterioration due to screen encrustation can usually be corrected by acid treatment and re-development.

7. Wellpoints should be installed as close to groundwater level as possible to permit maximum drawdown.

8. Header pipes should be kept level to avoid high spots where air can accumulate. Air separation chambers may be required if high points are unavoidable.

9. All pipe joints should be tight to avoid air leaks.

10. Wellpoints and ejectors should be sealed at ground surface, if possible, to aid in development of a vacuum.

11. Header corrections should be vacuum tight. When wellpoints are shutoff, the vacuum at the end of the system should be nearly the same as at the pumping station.

12. Dual pumping units are desirable so that routine maintenance can be performed without interupting pumping. When electric pumps are being used a standby diesel pump or diesel generator should be provided.

13. Initial wellpoints should be tested at varying depths to locate zones of highest yield. Subsequent wellpoints should be spot-tested to verify that they are installed in that zone.

14. Ejector pipes must be sized to limit friction losses.

15. Strainers should be provided on the supply side of each ejector to prevent clogging of nozzles.

4.20 RECHARGE SYSTEMS

A recharge well is always less efficient than a pumping well of the same design. For example, a vacuum is often produced at the well head as the water drops in the well inlet pipe. This drop in pressure allows gases to come out of solution, forming bubbles entrained in the water, which can clog the screen and filter. Therefore, even for the best designed recharge wells, two to three times as many recharge wells are typically needed as compared to the number of pumping wells required for a dewatering system. Additional considerations in the design of a recharge systems are described below.

4.21 Design

A filter is usually required in a recharge well. In addition, the screen slot size should be such that more than 90% of the filter material is retained by the screen.

The recharge head (the opposite of drawdown for the pumping well) is limited to the distance from the static water level to the ground surface, or the bottom of the lowest basement. If this head limit is less than that required, as calculated in the initial design, additional wells will be needed in the system.

To minimize well loss, the screen section should be as long as possible. To minimize air entrainment, the conductor pipe outlet must be below the static water level in the well, yet above the screen section. It may not always be possible to satisfy requirements, perhaps leading to a reduction in screen length and a subsequent reduction in well capacity.

It is preferable to use municipal water for recharge because the quality is typically more consistent and chlorination inhibits the growth of bacteria. When using municipal water though,

the possibility exists that an upset in the system, such as increased demand due to fire fighting needs, can flush iron and other particulate matter into the system. Whatever the source of recharge water, steps should be taken to minimize screen clogging by:

1. Reducing the amount of particulate matter in the water by settlement ponds or tanks, or possibly rapid sand filtration.

2. Reducing the potential for air entrainment and precipitation of dissolved solids by minimizing extremes in pressures and allowing for air to be vented at elevated positive pressure points (the header line and well casing) and assuring air does not enter at negative pressure points (at the well head).

3. Reducing the potential for bacterial growth by chemical treatment. Bacterial growth is enhanced when the injection water is warmer than the groundwater.

Each recharge well should have the capacity to operate as a pumping well to allow for the periodic backflushing of the screen and filter to remove clogging air bubbles and particulates.

4.22 Construction

Some considerations in construction of recharge systems follow:

1. The recharge well should have a screen size and filter size compatible with the natural formation material present (see Section 4.12).

2. The length of the well screen should be such that high exit velocities are not developed.

3. The wells should be provided with pumping capability so that periodic flow reversal can be performed to clear the screens and filter.

4. Installation of the recharge system at any distance less than twice the expected radius of influence will necessarily have an effect on the dewatering system design.

4.30 CUTOFF

Cutoff systems are typically not subject to design for purposes of water exclusion. Other criteria, such as lateral earth pressures or protection of adjacent structures, typically govern the selection process of a particular system. If however, groundwater exclusion is a controlling factor in the cutoff system design, the following items should be considered.

4.31 Steel Sheeting

While steel sheet pile walls are infrequently used purely for cutoff purposes, they are commonly used as part of a cofferdam structure to facilitate cut-and-cover construction. The effectiveness of steel sheet piling as a cut off is dependent primarily on soil conditions and water tightness of interlocks. The sheeting is most efficient in relatively coarse granular soils with high permeabilities where the flows through the sheet wall system are small with respect to the unimpeded flows through the soil mass. However, in these situations provisions may have to be made to handle the residual leakage which will develop through the sheeting interlocks and under the wall. In extreme cases, the gradient beneath the sheetpile wall may be great enough to cause instability at the base of the excavation.

Other practical construction considerations are listed below:

1. A sheetpile wall system may not be an effective cut-off in a fine grained, low permeability strata.

2. To be most effective, the sections should be driven into an impervious stratum such as clay, glacial till, or weathered rock.

3. If the system cannot economically be installed to intercept an impervious stratum, a partially penetrating system can be used provided that there is an adequate factor of safety against bottom instability (see Figure 11).

4. Sheeting cannot be driven through obstructions such as utilities, old foundations, or boulders. In urban areas a 10 to 15 foot (3 to 4.6m) deep trench is often dug along the sheeting line to eliminate obstructions. The trench is usually backfilled with loose sand prior to driving.

5. Access for the sheeting installation rig is required at the surface.

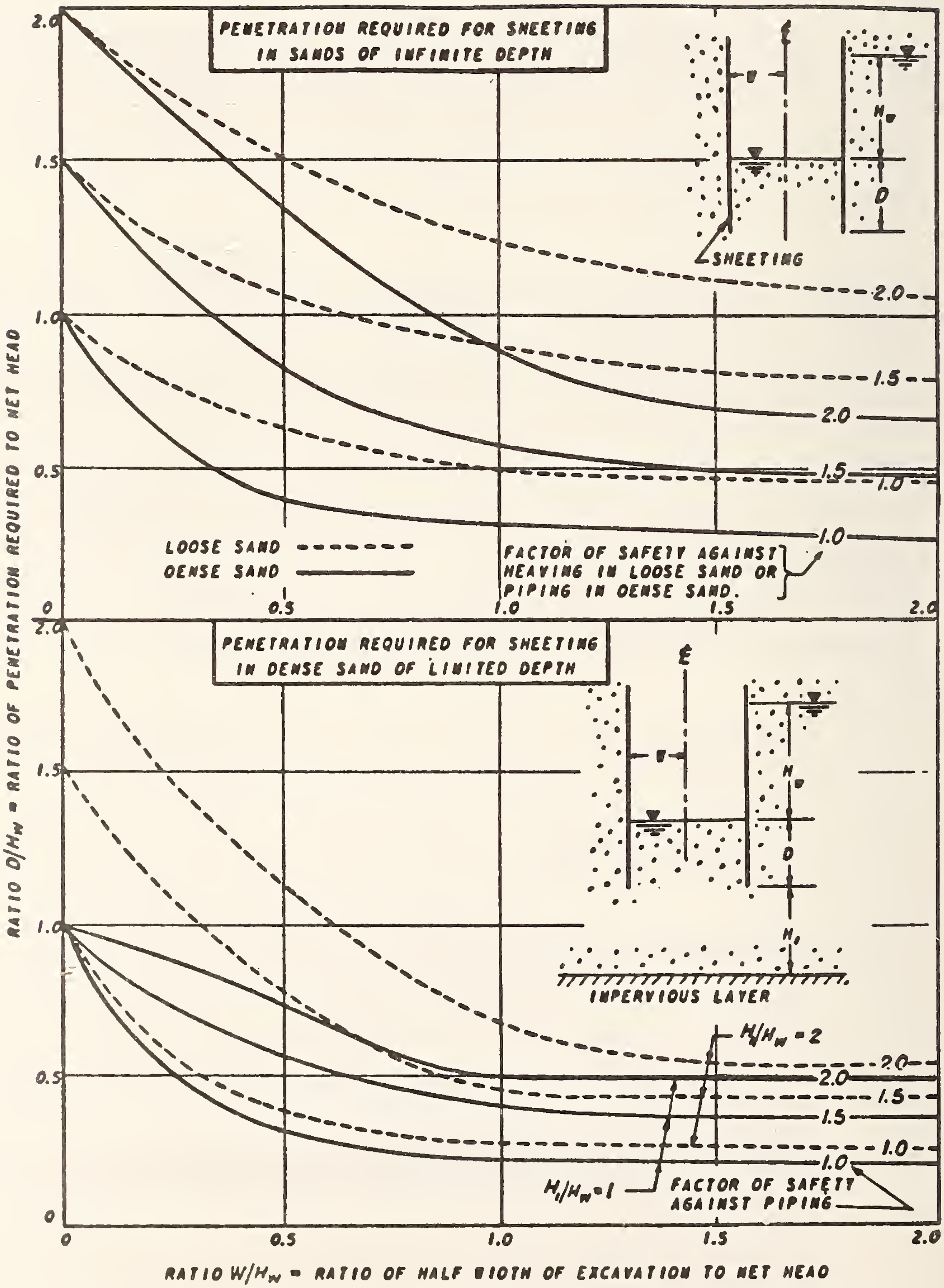


Figure 11 - Penetration of Sheeting Required to Prevent Piping in Isotropic Sand (Ref. 195)

6. Noise restriction may govern the method of pile installation in urban environments. A vibratory hammer may be required to reduce noise levels.

7. If leakage through sheetpile walls becomes too great (Figure 12;), as often happens after initial drawdown, measures may be required to reduce the flow. These may include "cindering" with fine-grained particles to clog the interlocks or stressing of the wall to induce tension at the interlocks. Over a period of time, water flows may be reduced entirely due to the corrosion of steel or natural migration of soil particles into the interlocks under flow conditions. The results of experimental and empirical work on the seepage through intact interlocks indicates that, as an approximation, it may be assumed to be on the order of .01 gallons per minute per square foot of wall per foot of differential head.

8. The steel sections will be subject to corrosion, especially in adverse chemical environments. Since the sheeting is often reuseable it may be protected in long term jobs by painting or coating, thus prolonging the life of the steel.

9. When driving into a dense impervious stratum, large penetrations to prevent underseepage may be difficult to achieve. Over-driving in tills, or weathered rock, can result in sheeting buckling and interlock damage.

4.32 Diaphragm Walls & Cutoff Trenches

The use of bentonite slurry is an innovative method for providing continuous impervious barriers within soil or rock. Precautions to insure wall impermeability depend greatly upon design variables as well as a number of construction related parameters. The basic sequence used in the construction of typical slurry walls, slurry trenches, and thin wall cutoffs are depicted schematically in Figures 13 through 15.

Waterproofing capabilities of diaphragm walls, are controlled by panel joint details which are the most likely sources for seepage. Figures 16 and 17 illustrate several possible joint details for reduction of seepage through cast-in-place and pre-cast slurry walls. The treatment of panel joints can often be accomplished using keyed joints, tongue-in-groove joints (pre-cast panels), waterstop joints or joint grouting. Since the concrete wall itself is relatively impervious if properly constructed, the design of the slurry wall system may be based upon minimizing joint spacing. The choice of joint treatment will depend greatly upon the local experience of the engineer and/or contractor.

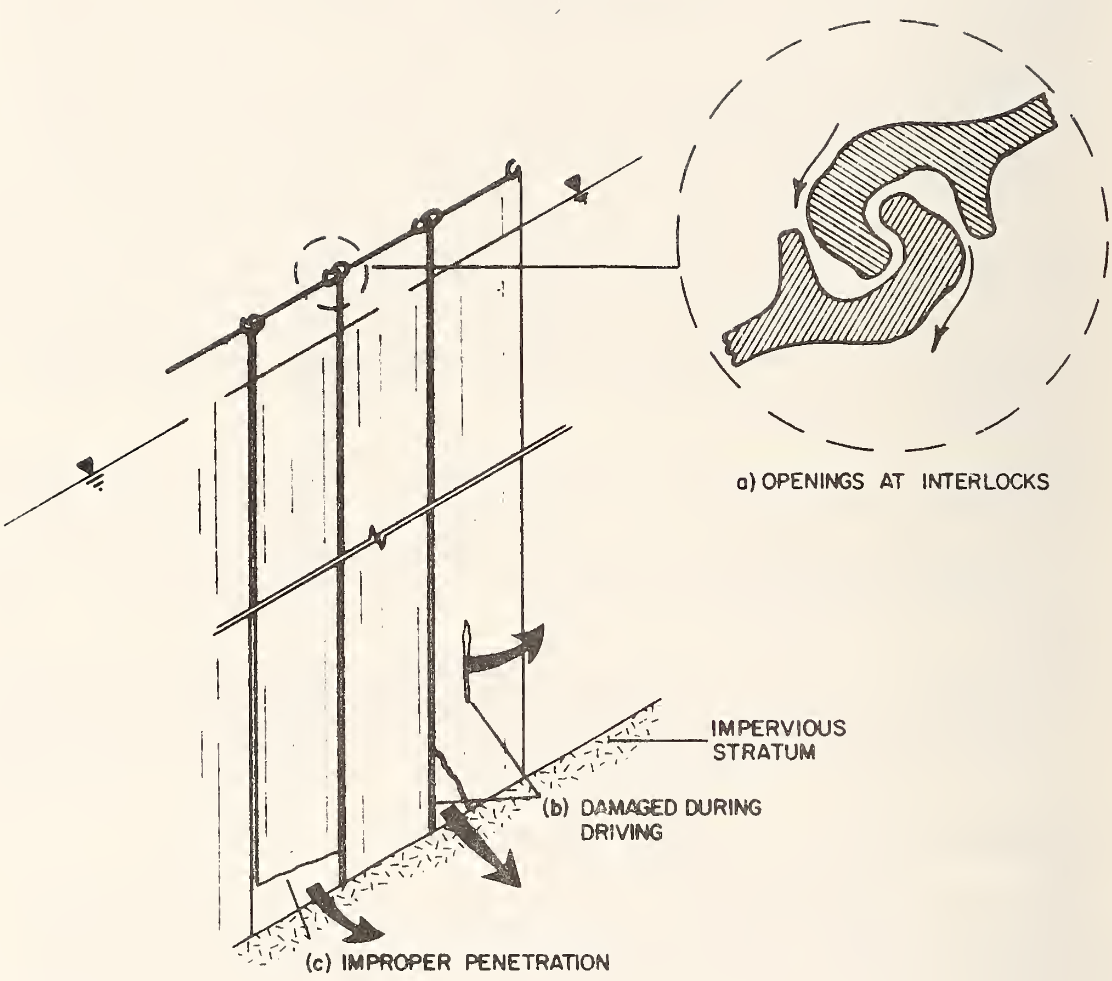
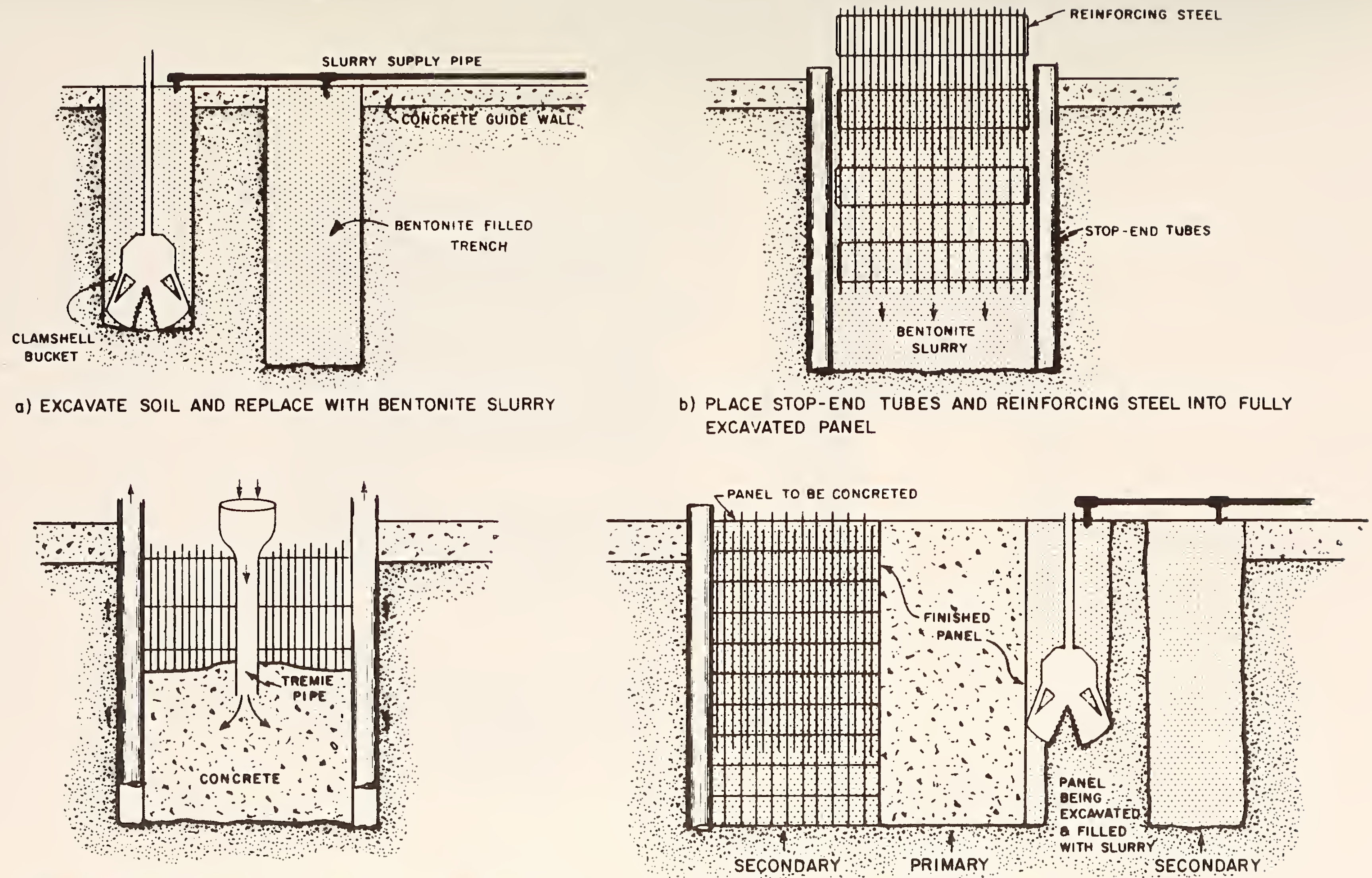


FIGURE 12 - Sources of Leakage Associated with Steel Sheet Cutoffs



a) EXCAVATE SOIL AND REPLACE WITH BENTONITE SLURRY

b) PLACE STOP-END TUBES AND REINFORCING STEEL INTO FULLY EXCAVATED PANEL

c) POUR TREMIE CONCRETE TO DISPLACE SLURRY, REMOVE STOP-END TUBES

d) DIFFERENT CONSTRUCTION PHASES

FIGURE 13 - Schematic of Conventional Cast-in-Place Slurry Wall

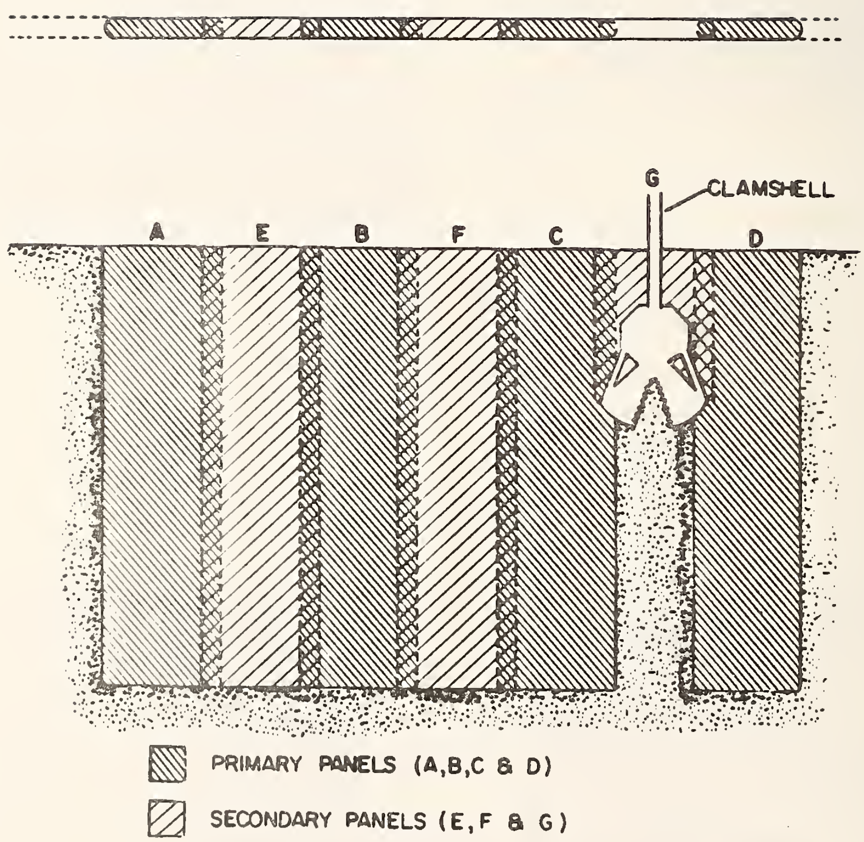


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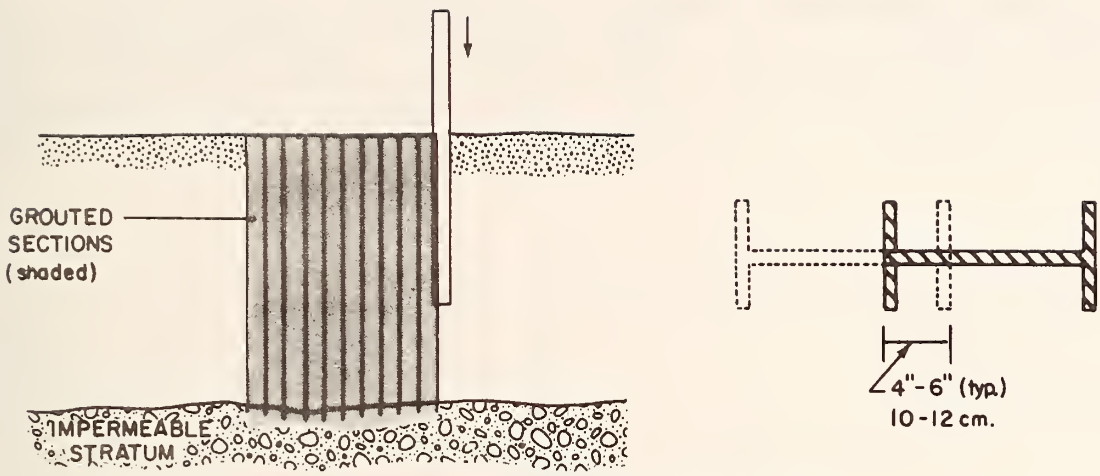
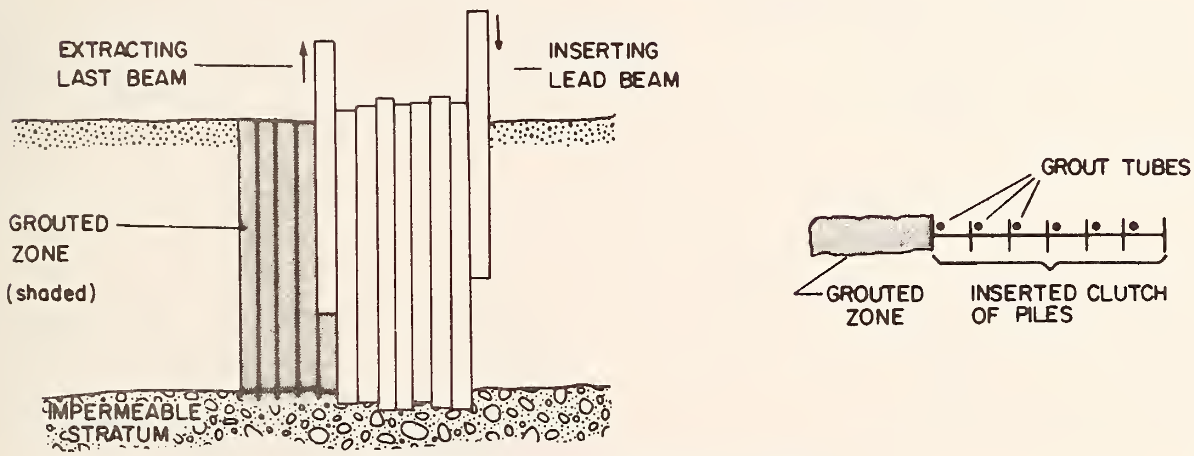
FIGURE 14 - Slurry Trench Panel Sequence



(a) INSERTION OF SINGLE INJECTION BEAM



(b) USE OF MULTIPLE INJECTION BEAMS

FIGURE 15 - Injection Beam Grouting (Ref. 57)

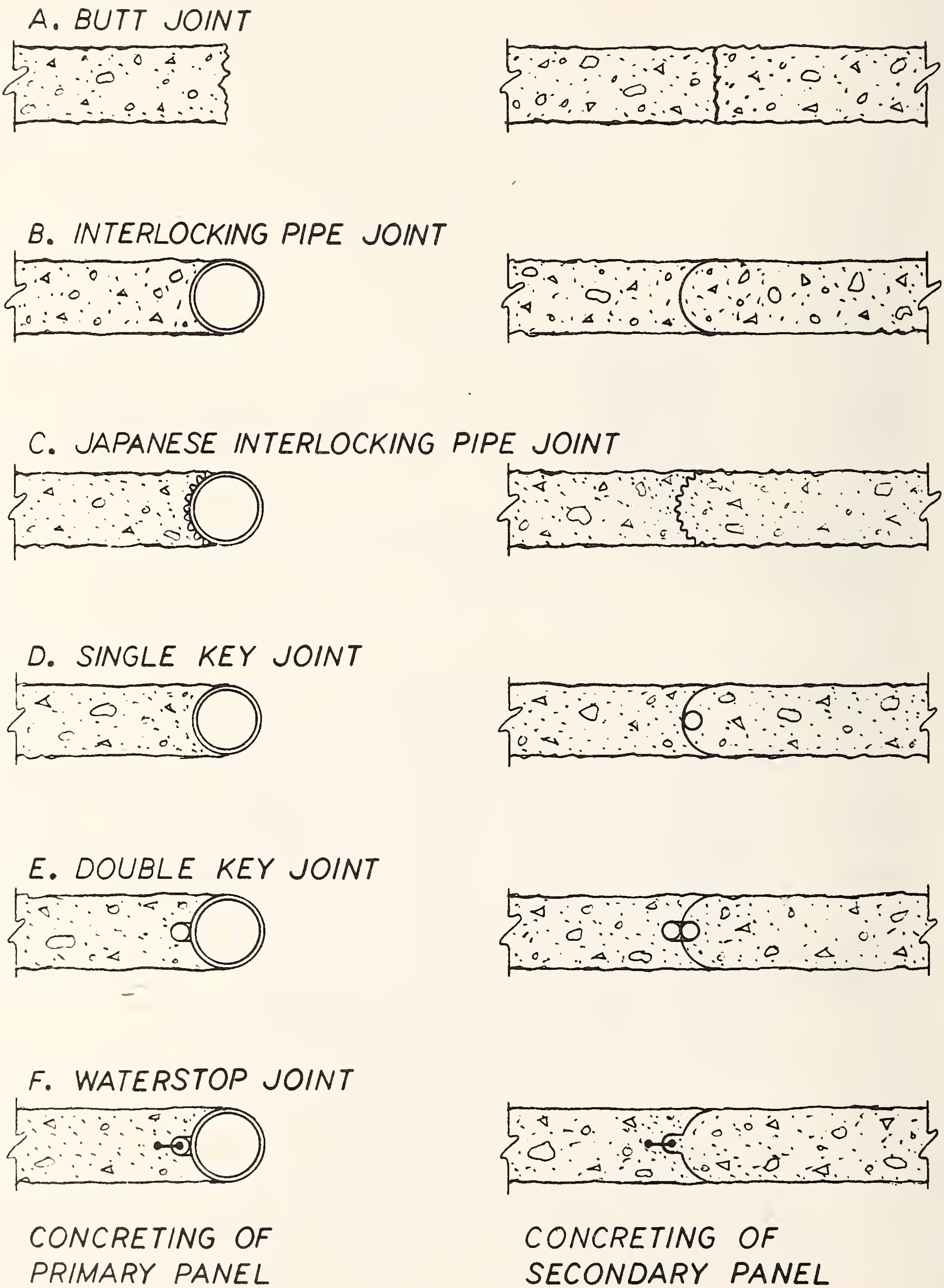


Figure 16 - CAST-IN-PLACE SLURRY WALLS - TYPICAL JOINT DETAILS

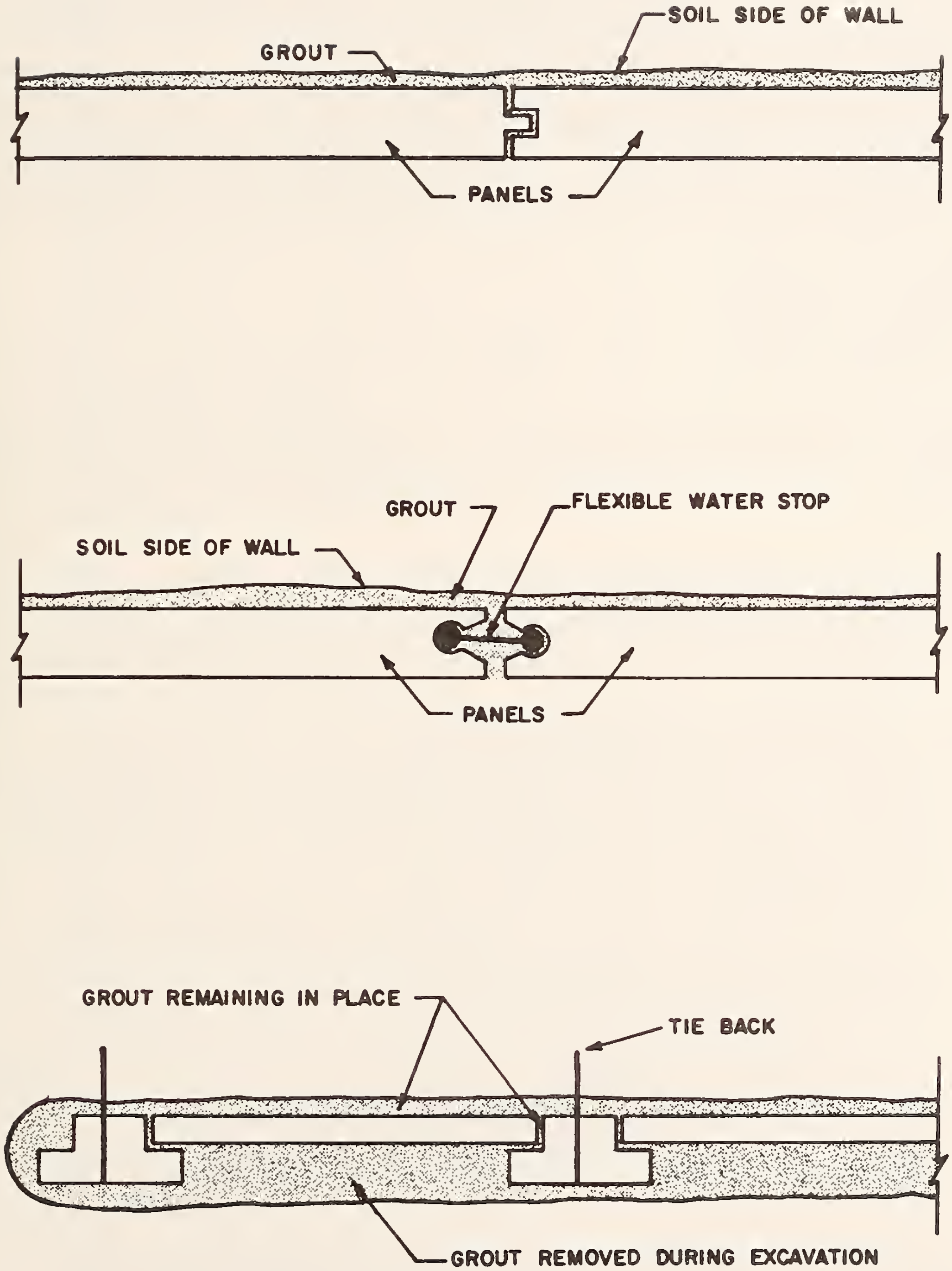


Figure 17 - Pre-Cast Slurry Walls - Typical Joint Details

4.33 Slurry Trenches

Slurry trenches of bentonite or bentonite/earth are typically continuous and thus not susceptible to seepage through joints as are slurry walls. Although feasible, they have not been used in connection with construction of tunnels in urban areas in the United States. Due to the limited experience with slurry trenches in the United States, the design may be based upon experience or trial and error. Reported thicknesses of slurry trench seepage barriers have varied from 1.5 feet (0.5m) (cement-bentonite cutoffs to 10 feet (3m) (soil-bentonite cutoff below embankment dams).

4.34 Thin Wall Cutoffs

Thin-wall cutoffs are constructed by grouting in voids created by driving wide-flange injection beams, (Figure 15). They are typically less than 1 foot (0.3m) thick. Design considerations include the grout consistency, injection pressure and beam spacing. Again, these may be chosen from experience or trial-and-error.

4.35 Additional Construction Considerations

Construction considerations relative to diaphragm walls, slurry trenches, and thin cutoffs are itemized below:

Diaphragm Walls and Slurry Walls

1. Access is required at the surface for the large panel excavation rig. In addition, a slurry storage area, mixing plant and de-sanding unit must be accessible to the trench.

2. A 2 to 5 foot (0.6m to 1.6m) thick guide wall must be constructed along the panel length to allow for excavation and alignment of the trench.

3. The bentonite slurry must be well mixed and of proper consistency, viscosity, and density to maintain trench stability and insure proper slurry displacement by the concrete. Precast panels require a special slurry mix which can be scraped from the inside wall face after excavation.

4. Provisions must be made to test the properties of the slurry throughout the panel depth. Test measurements include density, viscosity, shear strength, and pH. The contractor must be sure that the slurry has not

been contaminated by caving soil, calcium, salt, or other ground chemicals which may affect its performance. Table 9 summarizes properties of the bentonite slurry pertinent to slurry wall construction.

5. The slurry must be maintained at a level of 4 to 6 feet (1.2 to 1.8m) above the groundwater levels to provide a factor of safety against cave-ins and trench instability. High or perched groundwater levels may require dewatering in conjunction with the slurry head. The high head inside the panel also helps in the formation of the filter cake along the trench sides which is thought to be a major factor controlling permeability and stability of the slurry trench. Slurry loss in a very pervious layer must be compensated for by continued replenishing to fill the trench to required levels.

6. The area adjacent to the bentonite-filled trench must be capable of supporting heavy machinery such as cranes, compressors, and concrete trucks.

7. The finished panel typically includes a pre-fabricated cage of reinforcing steel which is usually fabricated on-site. This process requires extensive ground space and head room near the trench.

8. The excavated trench must be cleaned of debris trapped in the slurry or at the trench bottom. If the wall is to be keyed into an impervious rock stratum, the base must be cleaned out by compressed air or suction.

9. The tremie concreting method must be carefully controlled to insure proper slurry displacement and bond of concrete to reinforcing steel.

10. The panel joints must be carefully formed to reduce seepage. Grout, special joint configurations or waterstops may be required to control leakage.

11. Concreting of any panel should be continuous to avoid the formation of "cold joints" in the panel.

Slurry Trenches

1. Access at the ground surface is required for the excavating unit and slurry mixing plant. A separate mixing plant may be required to mix the excavated soil with additional bentonite or cement and bentonite for placement into the trench.

TABLE 9 - PROPERTIES OF BENTONITE FLUID PERTINENT TO SLURRY WALL CONSTRUCTION

PROPERTY	APPLICATION	METHOD OF MEASUREMENT	TYPICAL VALUES*
Density	Stabilizes trench Affect concrete placement	Mud Balance	< 1.1 gm/ml
Viscosity/Gel Strength	Stability Considerations	Marsh Funnel (viscosity only) Direct Indicating Viscometer Shearometer (gel strength only)	30-90 seconds 30-90 sec./1.4-10 N/m^2 1.4-10 N/m^2
Filtration	Prevents fluid loss Trench stability	Filter Press	------------
Sand Content	Alters slurry density Possible segregation in mix	Sand-screen Set	------------
pH	Affects Slurry Properties	Colorimetric (Test Strips) Electrometric (Glass Electrode Meter)	9.5 - 12

* Federation of Piling Specialists (FPS) recommended values for bentonite fluid supplied to the trench in average soil conditions.

2. The slurry and slurry/earth backfill should have properties adequate to maintain trench stability and impermeability of the final product. Field measurement of slurry properties may be required to meet job specifications (viscosity, density, shear strength, etc.).

3. The slurry must be maintained at a level of 4 to 6 feet (1.2 -1.8m) above the outside groundwater levels (see Note (5) above).

4. The set time of the backfill should be regulated to allow for continuous panel construcion without significant flowing of the previous day's backfill. Problems may arise with the use of a weak mix of soil-bentonite backfill.

5. The mixture and placement of soil-bentonite backfill for a cutoff must be well controlled to avoid the formation of "windows" of soil which will result in undesired seepage.

Thin Wall Cutoffs

1. Access is required for the crane, beam, and cement-bentonite mixing plant at the surface.

2. Adequate overlapping of the injection beam is required to reduce the possibility of windows of untreated soil in the cut off wall. This requires strict quality control and careful construction practice.

In general, considerations other than those listed may become important due to regional variations in equipment and material availability and local anomalies such as noise or dust restrictions, limitations of disposal of excavated soil and slurry, and availability of water and power. These considerations must be accounted for on a job-to-job basis.

4.40 GROUTING

Grouting is usually performed by specialty contractors. Grouting is a flexible process which permits effective response to variable and unforeseen conditions. The contractor must provide careful field control of the injection procedure including monitoring and interpretation of the rates, volumes and pressures within his grout system.

Once it has been determined that grouting may be required, the engineer or contractor must make a number of decisions including:

1. Choice of grout type

2. Determination of the zone to be stabilizea

3. Available access to the area to be grouted

4. Choice of injection method

5. Verification and monitoring of the grouted zone

Most of these decisions are usually made by the specialty contractor. A "performance" specification is usually provided by the engineer for the contractor to meet. Flexibility in design and specification of grouting systems may be the key to successful utilization of the grouting process.

4.41 Design

The following paragraphs detail some of the major issues which should be considered in the design of the grouting systems for groundwater control.

Choice of Grout Type

The grout should be compatible with the soil or rock to be stabilized. Among the major factors governing grout selection are:

1. The grout should be capable of permeating the soil mass to form an impervious barrier. A detailed discussion of the penetrating abilities of various grouts is contained in Volume 1 and summarized in Tables 10 and 11 for various soil and rock types. Figure 18 illustrates the ranges of viscosity which may be expected for some chemical and bentonite grouts.

2. The durability of the grouted zone may become important if a grout is intended to waterproof for the life of the tunnel. Certain grouts are susceptible to long-term degradation, the rate of which depends upon environmental factors such as temperature, soil pH, groundwater chemcials, etc. Each case must be considered separately.

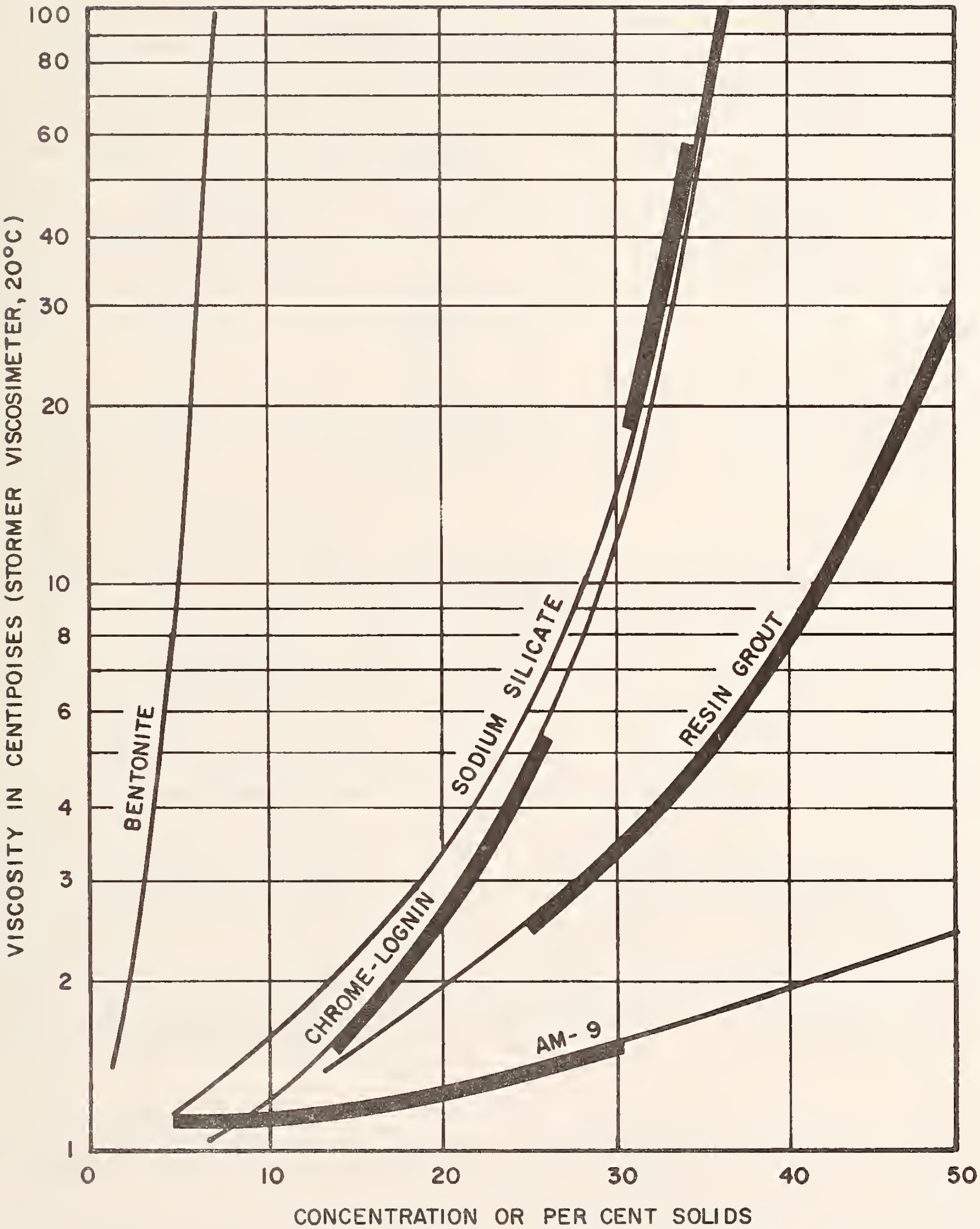
3. Proper adjustment of grout set time may required to insure adequate placement, especially under flowing groundwater conditions. Figure 19 summarizes setting times for chemical grouts.

TABLE 10-- GROUT APPLICATIONS IN SOIL

SOIL TYPE	TREATMENT	REMARKS	LIMITATIONS
Coarse Sands $k > 10^{-1}$ cm/sec	Clay-bentonite grout Clay-bentonite-cement grout Cement grout and chemical grout	Penetrability a function of grain-size of soil Primary treatments often used	
Medium-Fine sands $k = 10^{-1}$ to 10^{-3} cm/sec	Bentonite grout clay--chemical grout	Economical chemicals increase stability and accelerate set	Limited where large groundwater movements are present
	Silicate grout	One-shot process Cheapest chemical grout controllable set time Low viscosity Low permeability Two-shot process for very rapid set times	Possible long term degradation
	Lignin based grouts	Easily injected Reasonably inexpensive Controllable set time Low viscosity, permeability	Possible toxic effects
	Acrylamide (AM-9)	Controlled set time Very low viscosity Long term stability Low permeability	Very expensive Possible toxic effects Not much more effective than silicate grouts

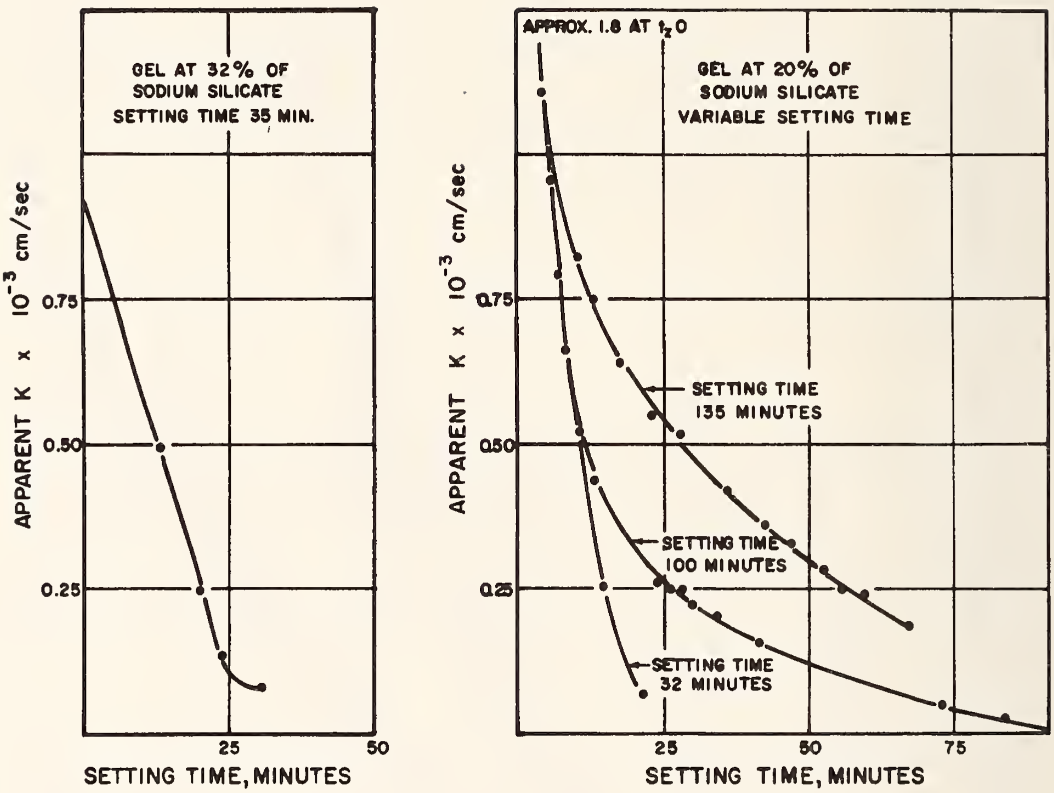
TABLE 11 -- GROUT APPLICATIONS IN ROCK

	TREATMENT	REMARKS	LIMITATIONS
Rock Fissures -large	Sand-cement grout Cement grout with fillers Hot asphalt Clay or bentonite-cement grout	Usually economical Penetration depends on grain size of grout components Admixtures adjust set time, flowability, stability	Possible segregation Limited where there are groundwater flows (except stability)
-small	Clay-bentonite cement grout Clay or bentonite grout Cement grout and chemical grout	Economical Admixtures adjust set time, flowability, stability Primary and secondary treatments often used	Possible segregation



Courtesy: Am. Cyanamid

Figure 18 - Viscosities of Various Grouts



a) SAND No. 1 b) CLAYEY SAND WITH k = 4.7×10^{-3} cm/sec

Figure 19 - Effect of Setting Time on Viscosity for Silica Gel (Ref. 256)

4. Components of some grouts are toxic. The existence of nearby water supply aquifers may preclude the use of these grouts due to possible contamination.

5. The relative cost of various grouts in-situ will vary according to geographical location, handling and injection costs, relative quantities required, and experience of local contractors. Table 12 summarizes some available data on the relative costs of various grouts in the United States and Europe.

Determination of the Zone to be Grouted

Some of the practical considerations which should be addressed prior to grout injection are:

1. The grout must be placed in the proper locations around the tunnel or excavation. In non-homogeneous profiles, the use of packer grouting or the "tube-a-manchette" (Figure 20) process will increase the accuracy of grout injection.

2. The treated zone must withstand high groundwater gradients, potential grout deterioration, and ground movement. The thickness of the treated zone may be specified arbitrarily or from experience with particular types of grout.

3. Provisions should be made for remedial grouting if seepage should occur during excavation, particularly in areas with erratic soil conditions.

Figures 21 through 30 illustrate grouting schemes utilized on several European projects, as well as some hypothetical excavations. If the grouted zone is to serve any purpose other than water exclusion, (such as underpinning or soil strengthening), the thickness of the treated zone will be governed by other criteria.

4.42 Construction

Access

Considerations relative to how the zone is to be grouted is to be physically reached include:

1. Space is required for grout storage, mixing, and pumping equipment. The size of this space will vary from something as small as a few square feet to areas of several thousand square feet.

TABLE 12 -- RELATIVE COSTS OF GROUT MATERIALS (Refs. 128, 185, 237, 253)

	RELATIVE COST FOR MATERIALS			RELATIVE COST IN PLACE		
GROUTING MATERIAL	U.K.	FRANCE	U.S.	U.K.	FRANCE	U.S.
Cement-Bentonite	1		-- 1		--	1
Deflocculated Bentonite	1.8		-- 1.8	1	--	1
Cement	--		1.0 4.2		1	
Silicates						
-one-shot dilute	3.3-7	2-4	1.3 --	1.2 to 14	1-2.6	1.4-2.7
-one-shot concentrated		6	2.9 --			
-two-shot	6	--	-- 10.7		--	
Lignosulfates	--	--	1.65 6.5-8		1.3-2.6	
Acrylamides	11-27	--	7.0 50-130		1.3-2.6	
Resins	--	10-500	6.0 10-40 250-500		--	
Bituminous Emulsion	--	--	-- 6-12	--	--	--

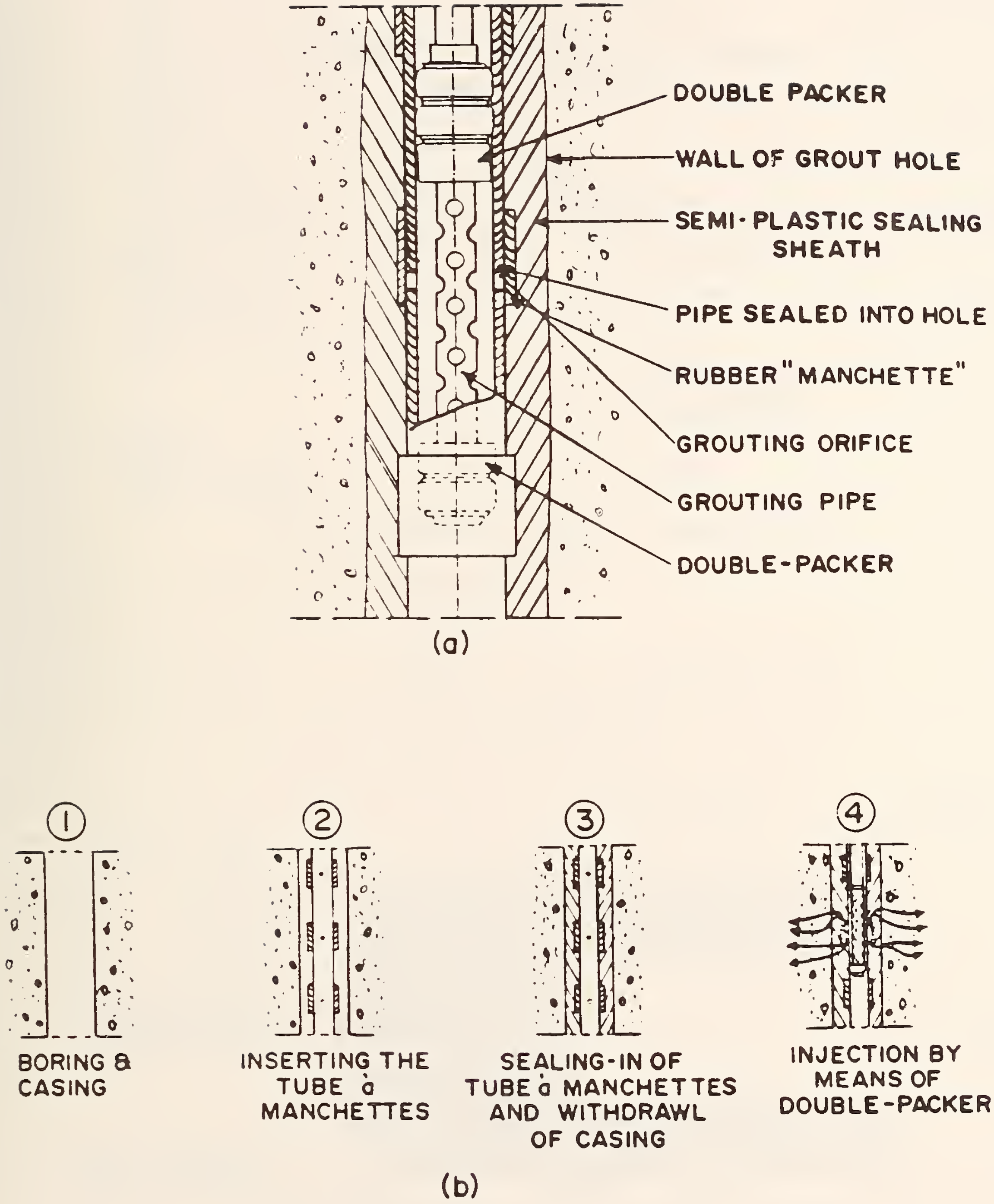


Figure 20 - Operational Principle of Tube-a-Manchette (Ref.133)

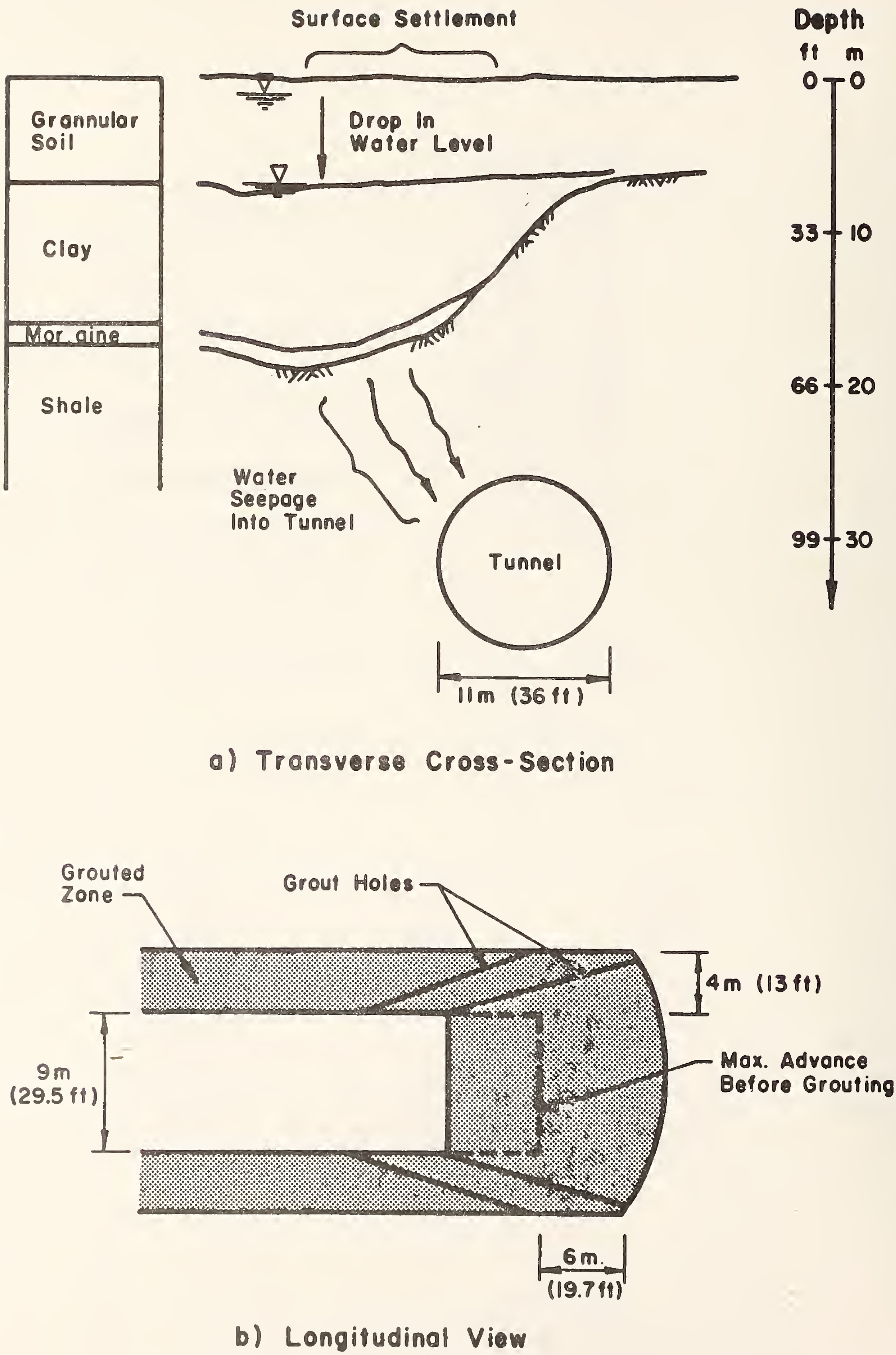
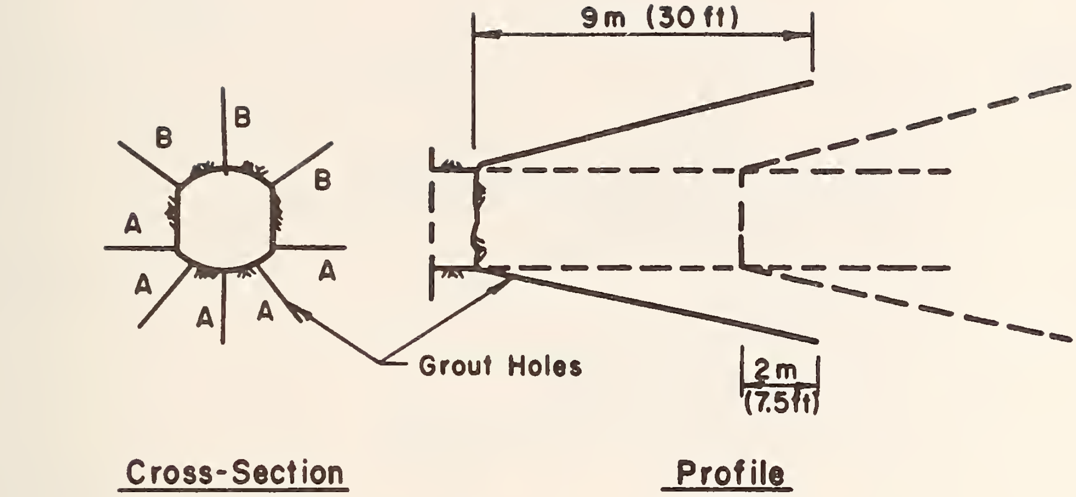
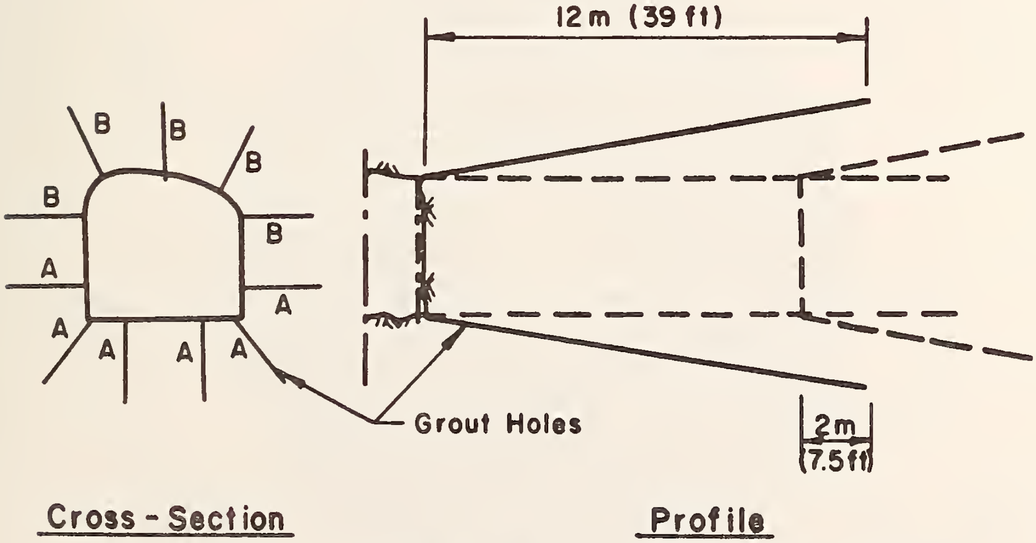


Figure 21 - Drainage and Rock Grouting Pattern for Rail Tunnel in Oslo (Ref. 201)



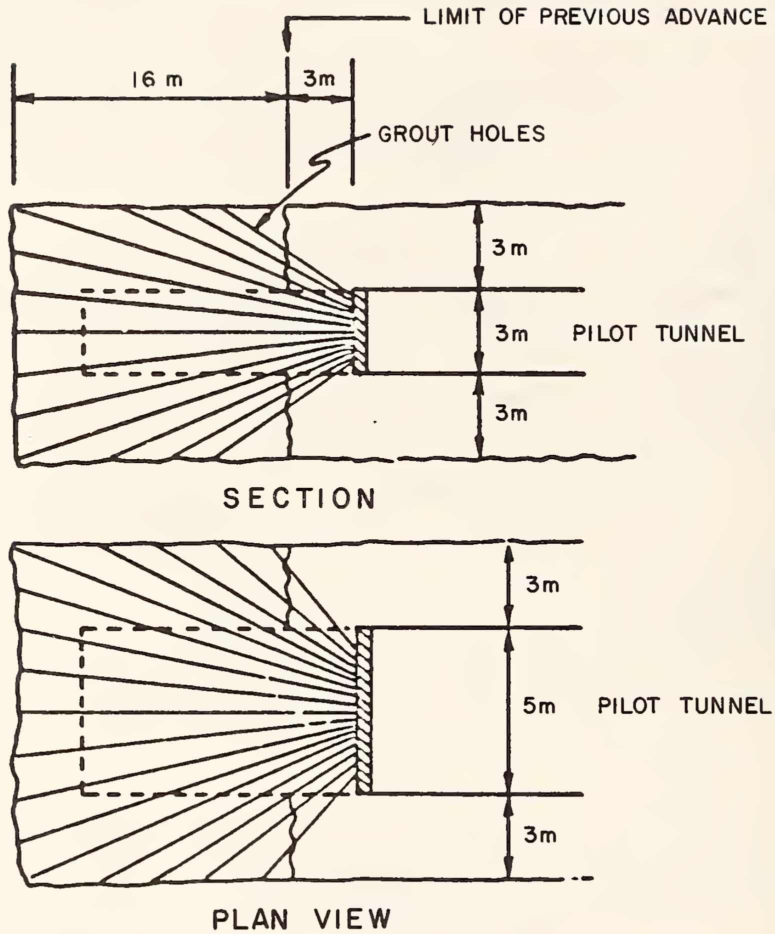
a) Tunnel Area ; 5-8 m^2 (54-86 ft^2)



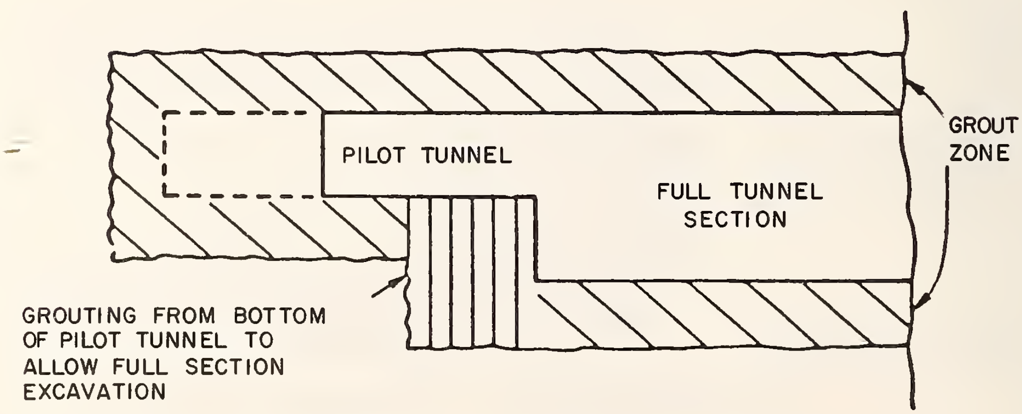
b) Tunnel Area : 9-17 m^2 (97-129 ft^2)

Note : A = Bottom Grouting
B = Full Front Grouting

Figure 22 - Rock Grouting Pattern for Tunnels in Gothenberg, Sweden (Ref. 201)



a) GROUTING OPERATION FROM FACE OF PILOT TUNNEL



b) SEQUENTIAL ADVANCE OF FULL TUNNEL SECTION

Figure 23 - Sequence of Grouting Operations Using Pilot Tunnel to Advance Tunnel Face (Ref. 71)

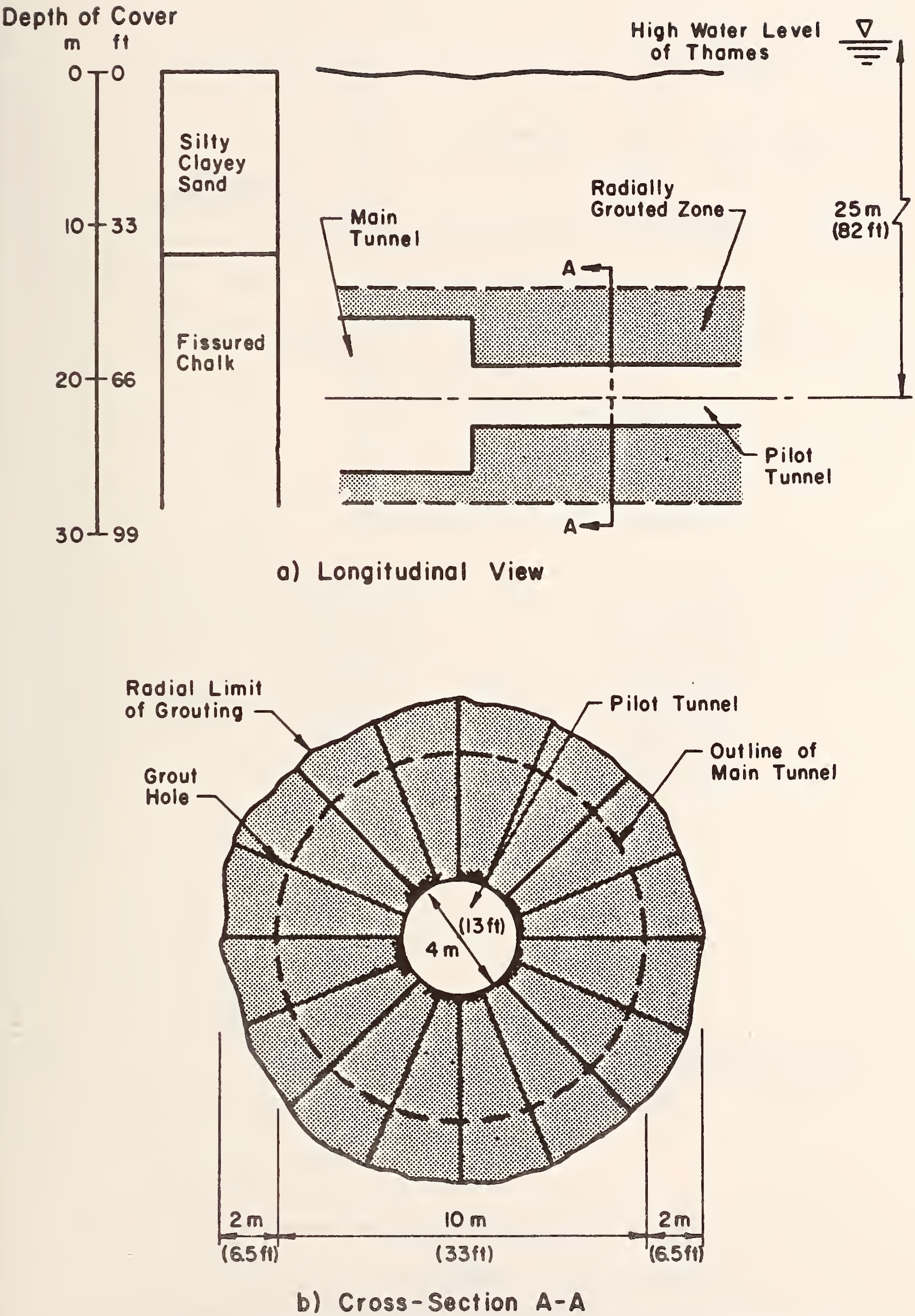


Figure 24 - Rock Grouting Pattern for Second Dartfort Tunnel (Ref. 201)

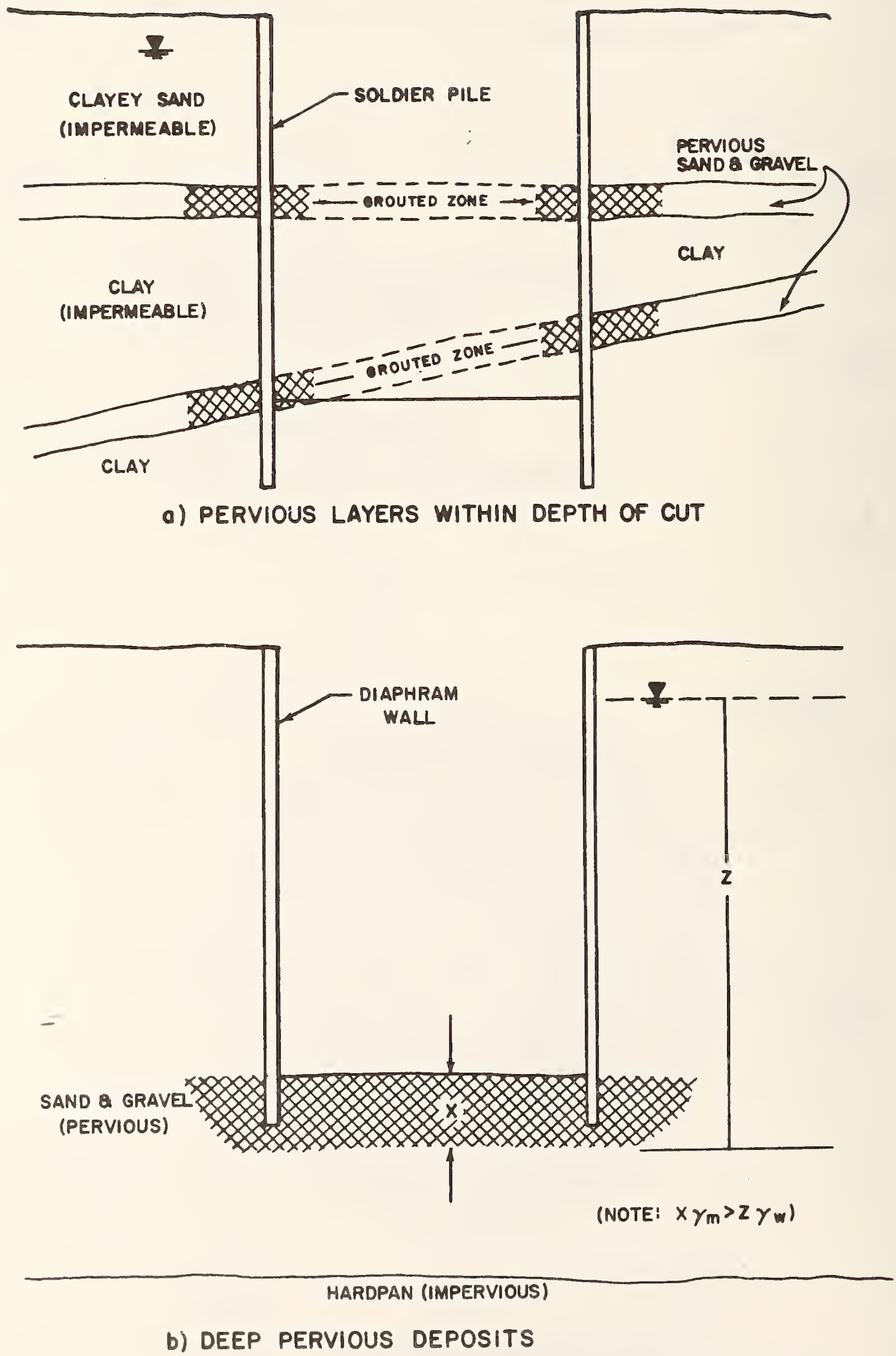
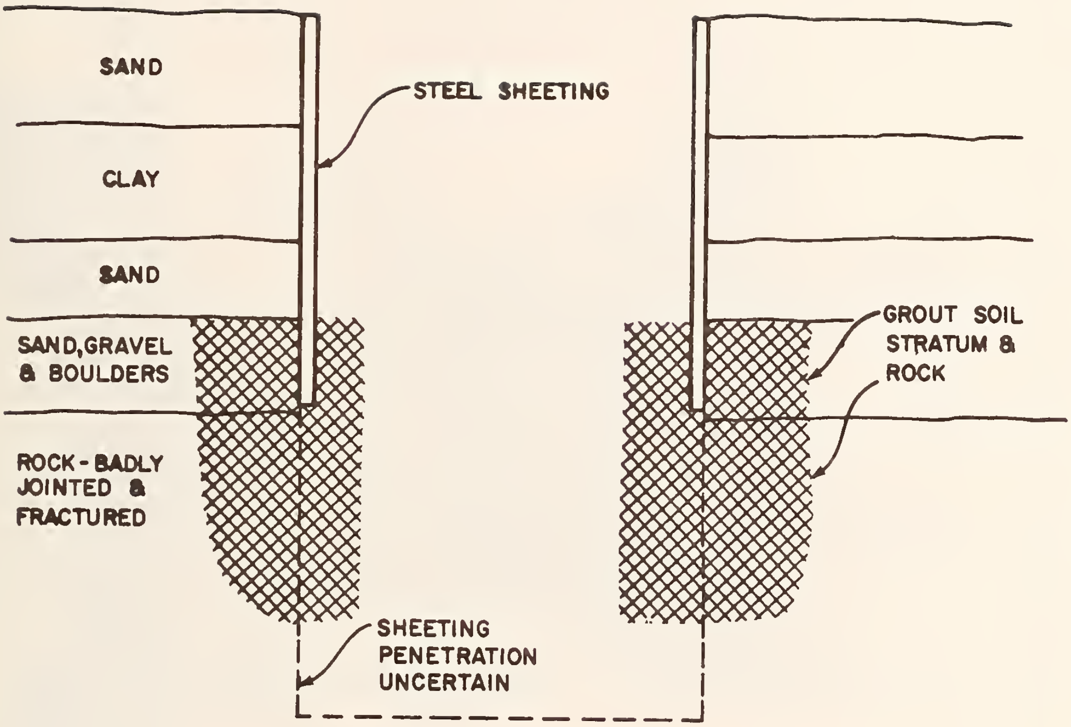


Figure 25 - Grouting for Groundwater Control in Cut-and-Cover Excavations



c) BOULDERS OR SHATTERED ROCK

Figure 26 - Grouting for Groundwater Control in Cut-and-Cover Excavations (cont'd)

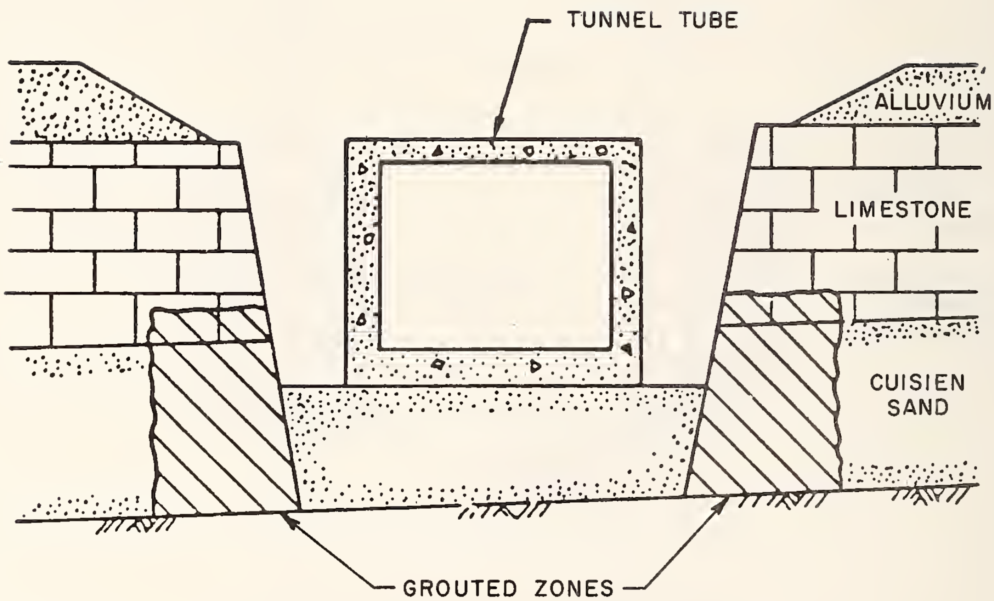


Figure 27 - Grouting of Cuisen Sand in Tunnel Tube
Bed, Paris Metro (Ref. 72)

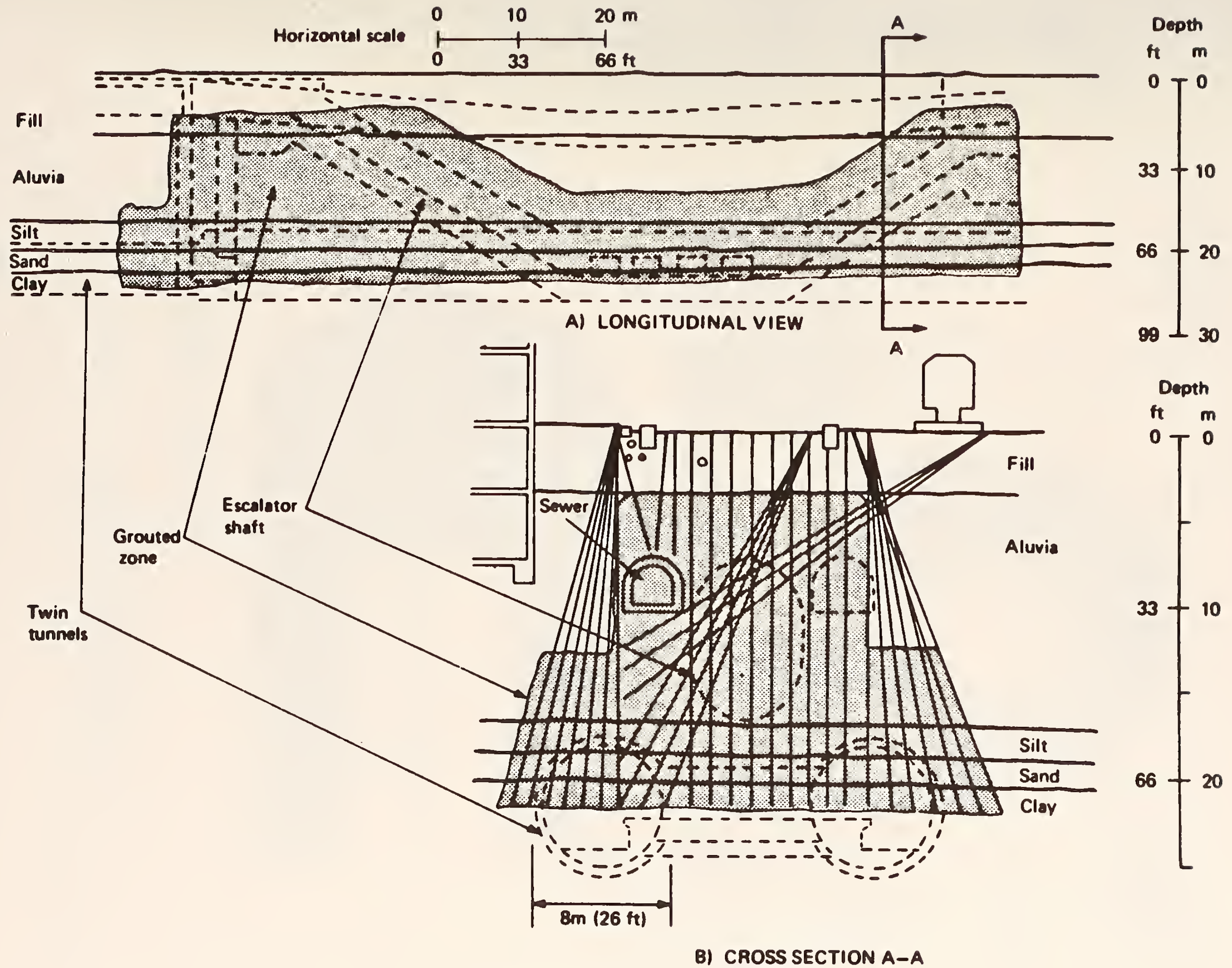
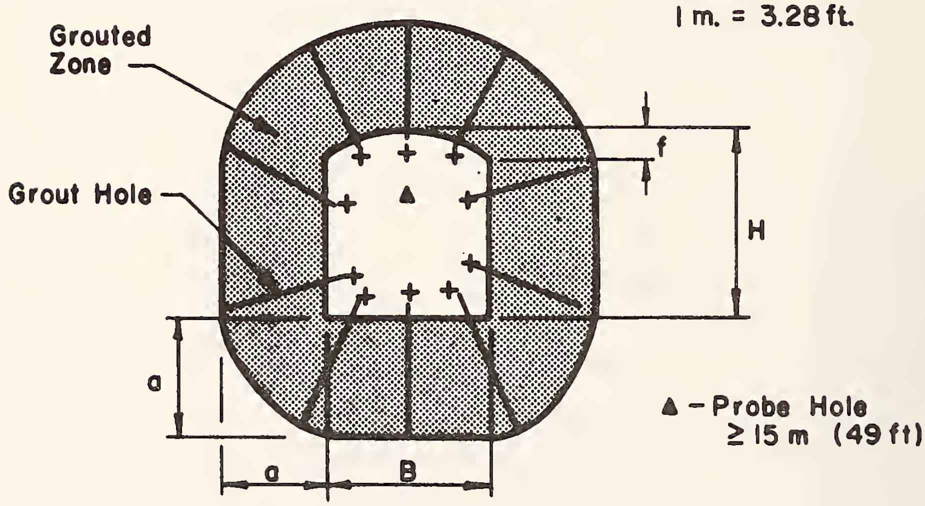


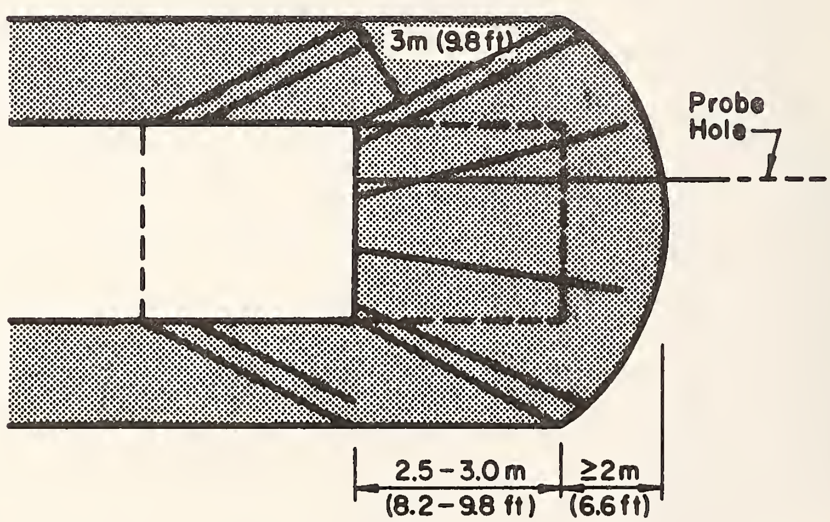
FIGURE 28 - Grouting Scheme, Karplatz Station, Vienna (Ref. 201)

Type	H	B	f	a
Station Tunnel	6.75	9.20	≥1.40	3.50
Double Track Tunnel	6.00	8.10	≥1.60	3.50
Single Track Tunnel	5.60	4.30	≥1.00	3.00

Note: All Dimensions In Meters 1 m. = 3.28 ft.



a) Transverse Cross-Section



b) Longitudinal View

Figure 29 - Rock Grouting Pattern for Tunnels Stockholm Underground (Ref. 201)

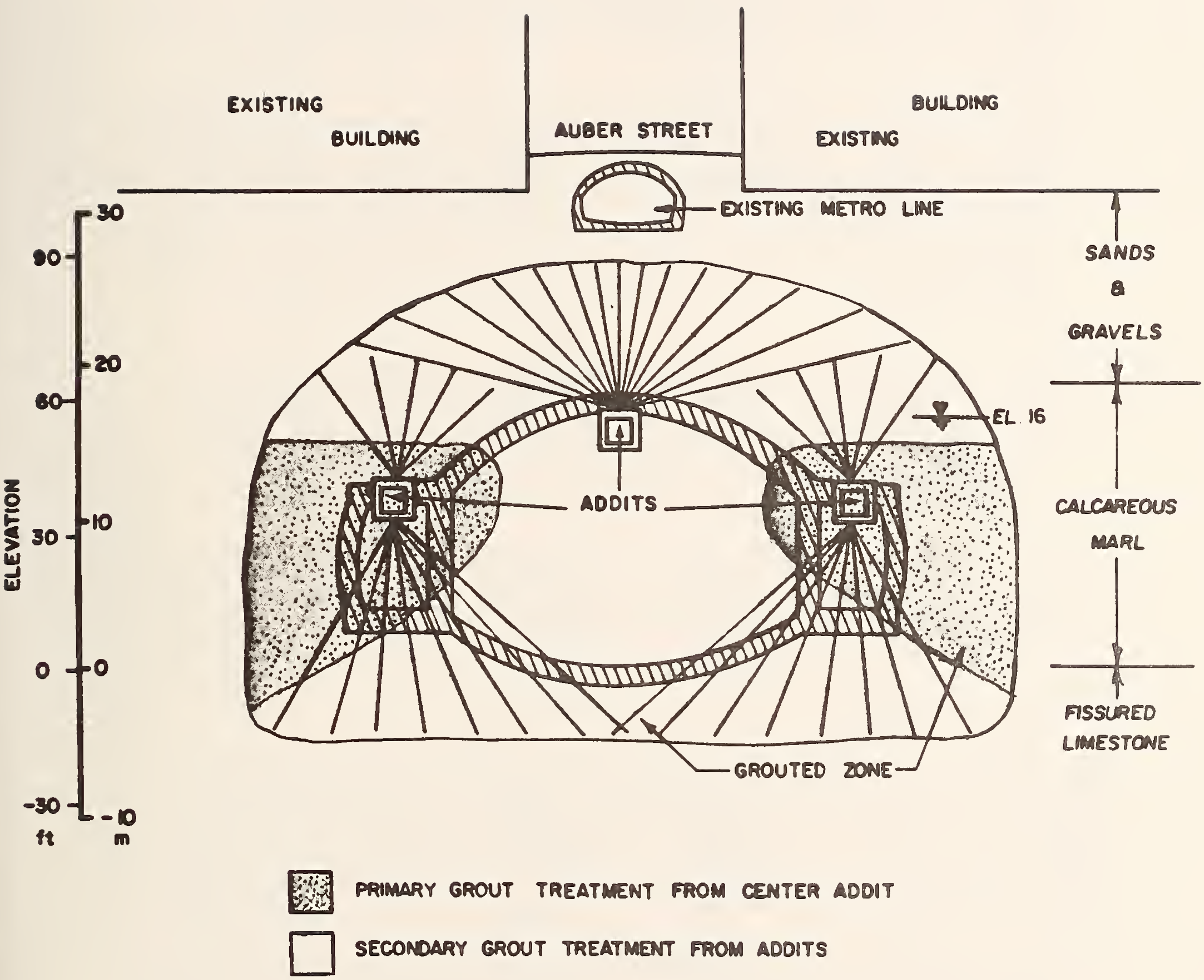


FIGURE 30 - Grouting from Galleries
Auber Station, Paris Metro (Ref. 133)

2. If grouting from the surface (primarily shallow tunnels), room must be made available for work crews, drilling rigs, and auxiliary equipment. This may prove to be a challenge in congested urban environments. Figure 28 illustrates a surface grouting scheme in Vienna, Austria.

3. Grouting from the tunnel face may pose unique problems due to limited space and difficult accessibility for machines and supplies. More importantly, actual excavation work may be held up by the grouting process, as two crews cannot work at the face simultaneously. Nevertheless, this approach is frequently used in urban areas as illustrated in the actual schemes of Figures 21 through 24.

4. On very large jobs, where grouting is impractical from both the surface and from the heading, special pilot tunnels or adits may be constructed and used for the injection of grout. Figure 30 illustrates a situation of this type used for construction of a portion of the Paris Metro.

5. If the point of injection is located at a significant distance from the grout mixing and batching plant, good communication lines must be established to allow for proper control.

Injection Method

The method of grout injection will vary with subsurface conditions and local experience. Table 13 summarizes some of the major techniques currently used, noting their advantages and limitations. A detailed description of these methods is included in Volume 1. Some other items to be considered include:

1. If stabilization is required to depths in excess of 40-50 feet (12.2-15.3m) or in very dense soil, a driven lance technique may not work.

2. The presence of stratified soils may mandate the use of a well controlled injection technique such as a packer or "tube-a-manchette" device (see figure 20).

3. In areas where re-grouting of a zone may be required (such as for a secondary treatment) it is advantageous to leave primary grouting holes accessible to avoid additional drilling costs. Figure 31 depicts a typical "split-spacing" layout for grout holes.

TABLE 13 Types of Injection Pipes

NAME	DESCRIPTION	PLACEMENT	ADVANTAGES	LIMITATIONS
A. Drive Rod (Lances)	EW Rod with special pointed end or extrudable plug.	Driven in ground	Pipe retrieved. Can grout in zones or stages.	For depths to about 50 feet. Requires driving equipment
B. Slotted Pipe	Plastic pipe with slots.	Set in borehole w/gravel & grout.	No special equipment required	Can grout only one zone.
C. Tube à Manchette (See Fig. 18)	Plastic pipe with sleeve covered holes at given intervals (French origin, available in Europe).	Set in borehole with weak grout. Uses inner pipe with packers to straddle holes.	Can grout selectively and regrout as desired.	May not be available in the United States
D. Lost Injection Element	Single section element with sleeve covered holes (Netherlands)	By special steel tube with small plastic hose to surface. Element is left in ground.	Simple if contractor has equipment required.	May not be available in U.S. Special placement vibrator crane is required. Can only grout one section about 1 meter thick.
E. Open Hole With Packer	Uses cased hole with air packer set in end of casing above zone to be grouted.	In borehole.	Can stage grout as desired.	Requires packer and air source.
F. Stabilator Valve Tube	Steel drilling system also used for grouting (Swedish).	Drills with pipe, then knocks off bit and uses pipe for grouting.	Can grout selectively.	May be hard to obtain in the United States.

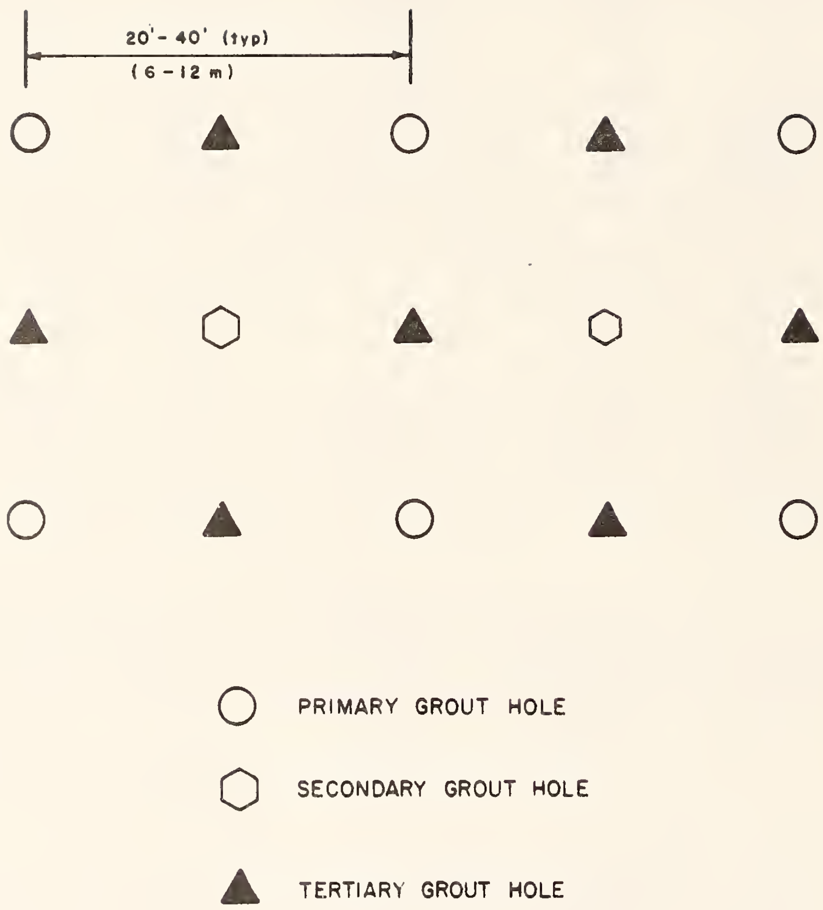


FIGURE 31 - Schematic of Split Spacing
Surface Application
(Ref. 76, 88, 89, 91, 104, 133, 140)

4.50 FREEZING

Ground freezing is a highly specialized process performed by relatively few contractors throughout the world. The design process is therefore, not unlike that for grouting, i.e. a specialty contractor is usually consulted. Description of some general design concepts follows.

4.51 Design

To design a ground freezing system, requires knowledge of:

1. Soil Permeability
2. Groundwater gradient
3. Frozen soil strength
4. Ground temperature
5. Freeze probe temperature
6. Soil Dry unit weight
7. Soil Moisture content
8. Frozen and unfrozen soil shear strength
9. Loading on the frozen structure

A minimum freeze probe spacing (typically 3-6 feet) (1-2m) is assumed which is equivalent to the design frozen soil wall thickness. Evaluation of the adequacy of the frozen thickness is based on the frozen strength of the soil and loading due to non-frozen soils using classical structural and geotechnical design methods. If the frozen structure is potentially unstable, the spacing may be adjusted to obtain an adequate factor of safety.

The required time for complete closure of a frozen section can be calculated based on maximum probe spacing (size of zone to be frozen) and coolant type as illustrated in Figure 32. Deviation from the design spacing of probe should be estimated based on local experience. Deviations of up to 5 to 10 percent of the hole depth are not uncommon. If freeze time is excessive and it is expected that the maxiumum spacing cannot be reduced, more efficient coolant types (liquid nitrogen, freon) can be used to reduce the freeze time, but at increased cost.

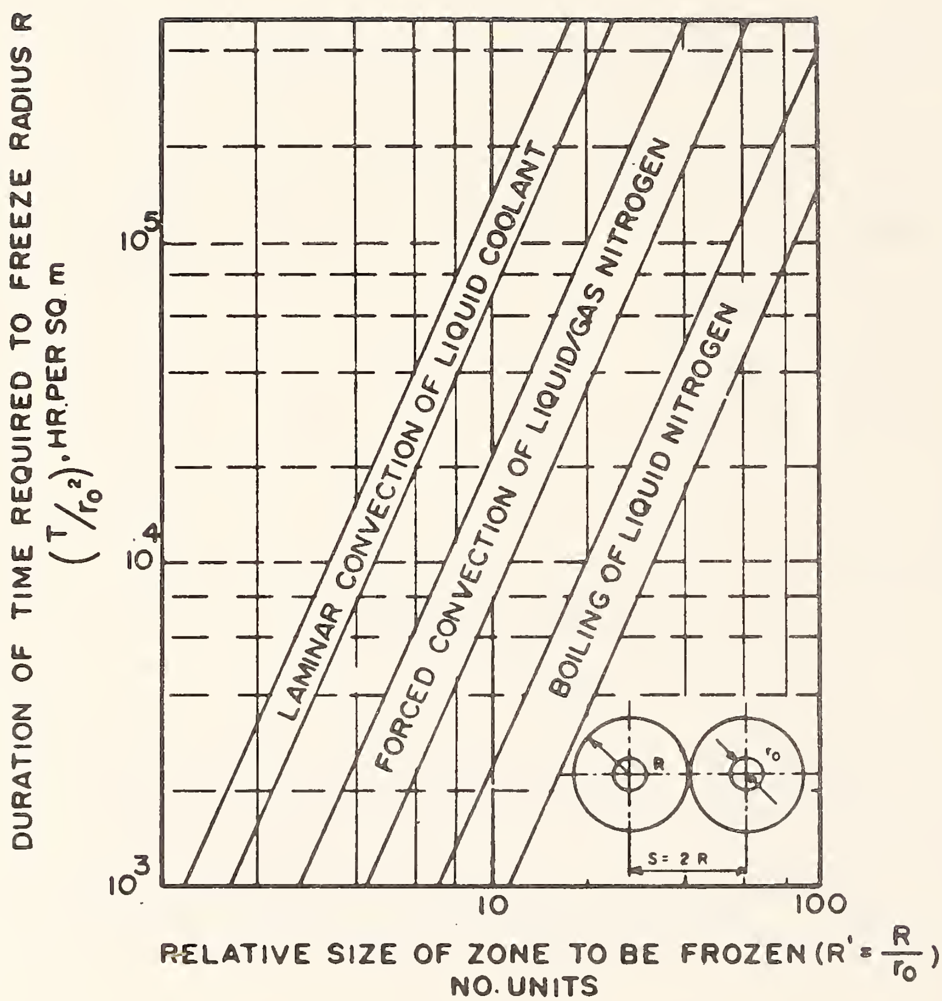


FIGURE 32 - Generalized Relationship Between Freeze Pipe Size, Spacing, and Time (Ref. 152)

For efficient subsurface heat transfer the velocity of the coolant through the annular space between the inner and outer pipes of the probe should be in the turbulent range. Therefore, select inner and outer pipe sizes such that coolant velocity at the selected flow rate will produce turbulent flow. Typical flow rates are 25-30 gpm (1.6 to 1.9 l/s).

The number of probes to be coupled in series and parallel to yield an acceptable system total flow rate and pumping pressure must be determined. In a series system, the return flow from a probe is used as the supply flow to the adjacent probe. Therefore, coolant temperature increases with each additional probe in the system, with a proportional drop in efficiency. Coolant pump requirements are for a low flow-rate, high pressure pump. In a parallel system, the flow through each probe is independent of the adjacent probes in that they are connected to a separate supply and return header. The input coolant temperature remains relatively constant to each probe. Pump requirements are therefore high flow-rate, low pressure.

In practice, a system is made up of a group of probes in series coupled to an overall layout operating in parallel. This allows for a more efficient pump system, operating within the extremes noted above, with minimal variation in coolant temperature in the series sections.

Finally mechanical design considerations are designed including the size of supply and return piping, coolant pump, freeze plant and power requirements (including an emergency standby system.

Figures 33 through 35 illustrated several examples of ground freezing for tunnels.

4.52 Construction

Some relevant construction considerations include:

1. Measurement of distance between adjacent probes should be made at time of installation. If spacing is beyond the maximum used in design, redrilling or the installation of additional probes may be required.

2. Couplings or welded joints of the outer freeze pipe should be pressure tested by air or water to assure that coolant leakage into the aquifer will not occur.

3. The inner freeze pipe should also have properly sealed joints, though no pressure testing is required. The bottom of the inner pipe should be approximately one foot (3.0m) above the bottom of the probe.

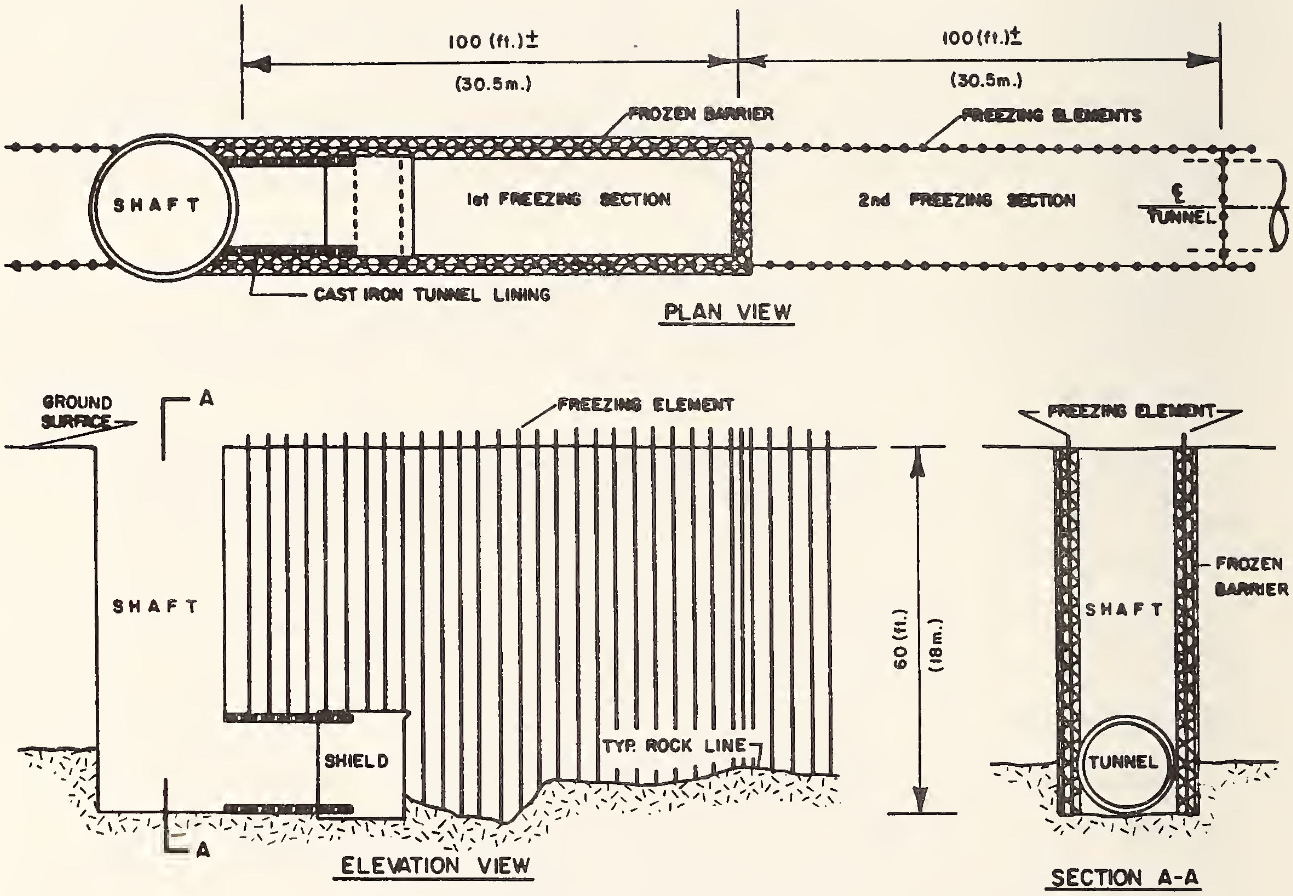
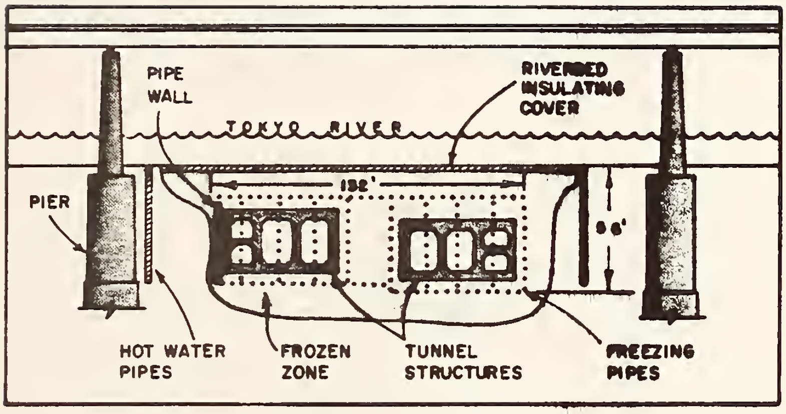


FIGURE 33 - Ground Freezing for Sewer Tunnel, New York City (Ref. 101)



NOTE: 1' = 0.305m

FIGURE 34 - Ground Freezing Under Tokyo River (Ref. 31)

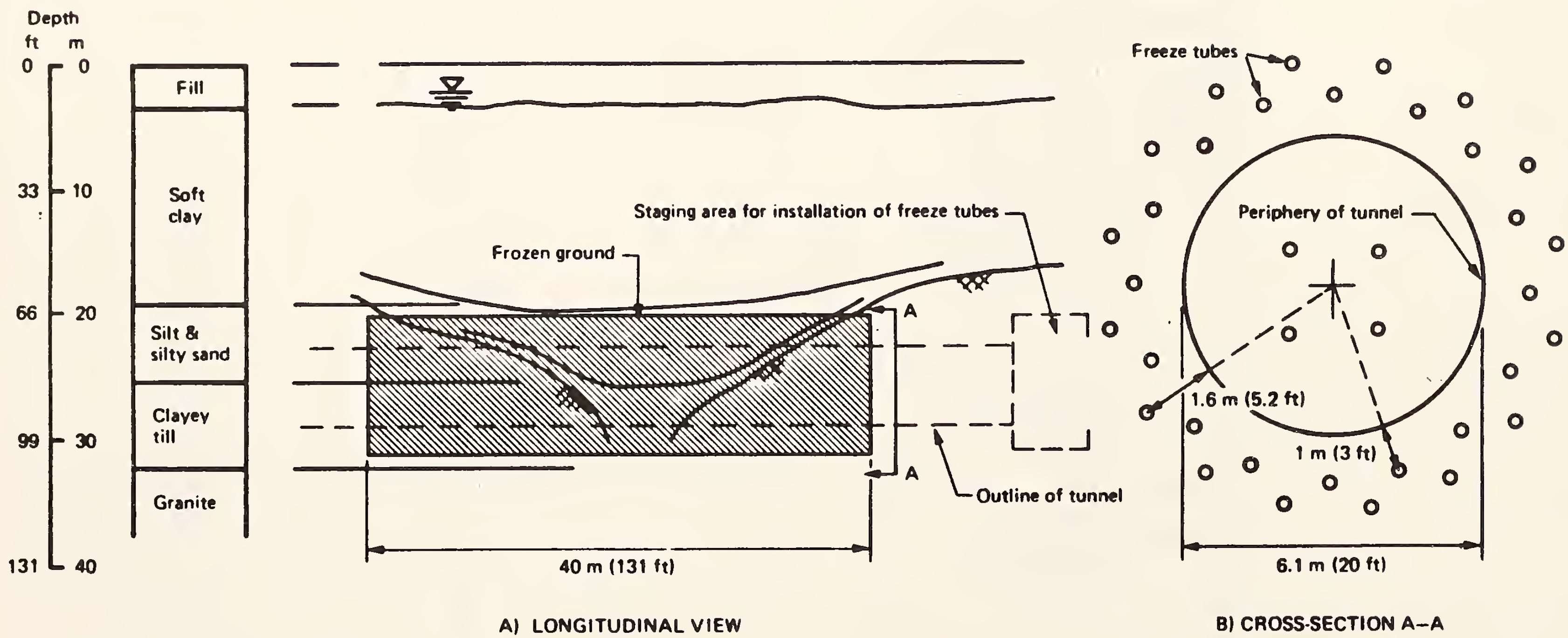


Figure 35 - Ground Freezing for Tunnels, Helsinki Metro (Ref. 201)

4. Adequate surface pipe insulation should be installed in accordance with ambient temperature range and surplus freeze plant capacity. Drape open excavations with heat reflective material where necessary to reduce solar effects on frozen soil surfaces.

4.60 COMPRESSED AIR

Compressed air tunneling is a highly specialized construction method that requires greater attention to details of construction and system maintenance for successful application than design principles.

4.61 Design

Design of a compressed air system for groundwater control, as opposed to maintenance of face stability in soft compressible soils, is relatively simple. Normally, the air pressure corresponds to the head at mid-height of the tunnel face. Design of the air locks and shield is a construction detail of which tunnel designers do not need to fully understand.

Location of air locks, whether in a shaft or the tunnel, is an important consideration to the planning of any compressed air tunnel. In general, tunneling is simplified with the lock located in the tunnel rather than a shaft, because movement of men and materials is easier on the horizontal than vertical. Placement of the lock in a shaft should be done only when all other groundwater control methods have proved to be infeasible.

4.62 Construction

Construction considerations for compressed air tunneling include:

1. Tunneling under compressed air is a more flexible procedure than other specialized shield techniques, i.e., slurry or earth pressure balance, because excavation is done by hand or mechanical excavator shield which can accommodate variable ground conditions more successfully than the complex specialty shields.

2. Productivity is reduced in compressed air tunnels because of reduced allowable working times. Table 14 is a summary of OSHA regulations concerning allowable working and decompression times versus operating air pressure. This is probably the single most significant construcion consideration relative to compressed air tunneling, particularly at pressures above 12 psi (83 k Pa).

TABLE 14-- SELECTED HOURS OF LABOR AND DECOMPRESSION TIMES

U. S. DEPARTMENT OF LABOR

SAFETY AND HEALTH REGULATIONS FOR CONSTRUCTION

DATED JUNE 24, 1974

Pressure psi	Pressure kPa	Hours of Labor	Decompression Time: Min.
0-12	0-83	8	3
16	110	6	33
		4	7
20	138	4	43
		3	15
24	166	4	92*
		3	52
28	193	3	98*
		2	41
32	221	2	85*
		1-1/2	43

*Decompression times in excess of 75 minutes require a special decompression facility

3. Muck handling is less efficient in compressed air tunnels than in free air, because materials must be passed through a locking system.

4. Although modern compressors are muffled quite effectively, their use in urban areas can still present problems as a result of 24 hour a day operation. Considerations must be made for noise control at the shaft area.

5. Care must be taken to control operating air pressures to prevent surface heave which commonly occurs, particularly in urban areas. Surface movements should be monitored to insure that surface disturbance, whether heave or settlement, is minimized.

6. In pervious granular soils there may be a need for back-up air compressors because of air loss.

7. It is essential to maintain backup equipment ready for immediate use to maintain air pressure in the event of compressor breakdown. Compressor breakdown could result in loss of the tunnel.

8. Compressed air tunneling in silt can be difficult because excess air pressure at the crown will drive water from the voids, thereby drying the silt until it cracks and may slough. Silt is very susceptible to unbalanced air pressures. It is frequently necessary to vary air pressure, such that when the arch is being mined a lower pressure can be used, and when the invert is being mined a higher pressure can be used.

9. Sand typically behaves in a similar manner to silt. However, it is more resistant to seepage pressures than silt and will remain more stable than will silt.

10. Compressed air tunneling through gravel is extremely difficult and it is often necessary to use clay to create an impervious zone within the tunnel to minimize air loss.

11. To conserve air in pervious ground, the lining should be kept as close to the heading as possible. It is often necessary to caulk or gasket, liner plated tunnels, to minimize air loss.

12. If a blow should occur, the normal procedure is to use any available material, whether it be a board or bag of cement, to plug the vent hole until the escape of air is checked. The hole may then be plugged to re-establish air pressure in the tunnel.

13. Blows can be fought from the outside on subaqueous work by dumping barge loads of clay. Sometimes it is necessary to place a blanket of clay along a river bottom over the projected route of a compressed air tunnel to minimize the possibility of a blow. Blows are more hazardous in short tunnels than long tunnels because a greater volume of air can escape from a long tunnel before pressure is lowered to dangerous levels.

14. Mayo, (Ref. 177) suggests the following rule in estimating the probable number and size of compressors. Provide 20 cubic feet (0.6 m^3) of free air per minute for every man in the heading. To these totals, must be added the amount of air which is lost with each passage of the lock. The above rules assume that the tunnel is practically air tight and that the lining is kept close to the tail of the shield as possible.

4.70 SLURRY AND EARTH PRESSURE BALANCE (EPB) SHIELDS

4.71 Design

The design of a slurry or earth pressure balance (EPB) shield is not a major consideration for a tunnel designer per se. The tunnel designer must understand what the shield will do and some significant limitations, however, he does not need to understand all of the structural and mechanical details of the system to arrive at a successful tunnel design. Therefore, discussion of details of design of slurry or EPB shields are not presented herein. Schematic illustrations of both types of shields are included as Figures 36 and 37.

A major consideraton relating to design of slurry shields is the maximum water pressure which can be withstood by the shield and successfully control ground movement. It is reported that maximum pressures are on the order to 2.0 atmospheres (202 kPa). This limiting pressure is not so much a matter of structural integrity of the shield as the ability of tail seals to withstand the necessary pressures to counteract external head. Tail seal details are illustrated in Figures 38 and 39.

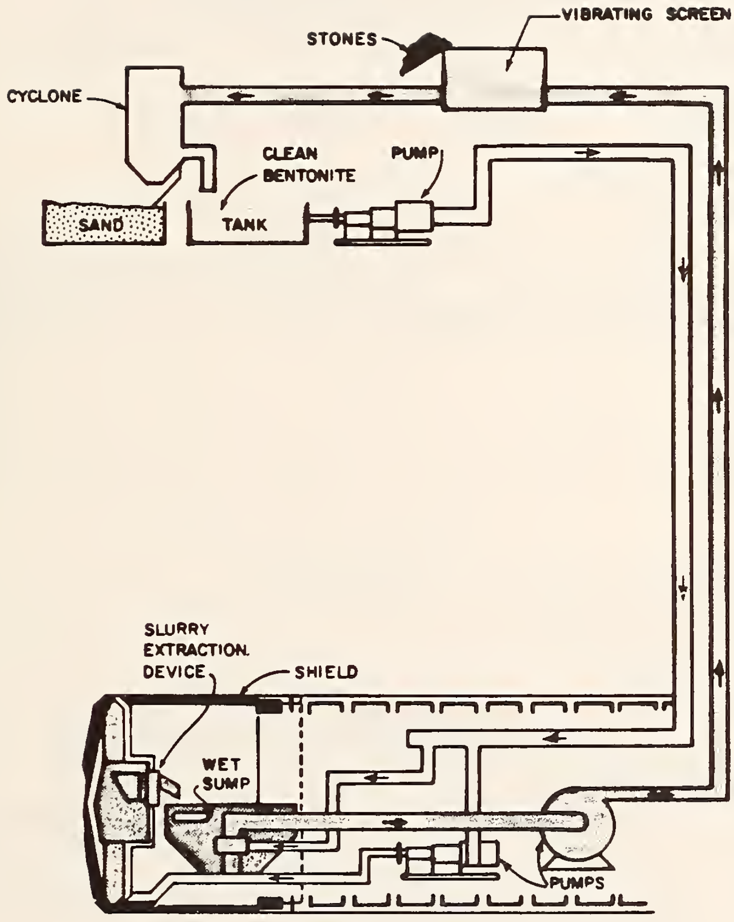
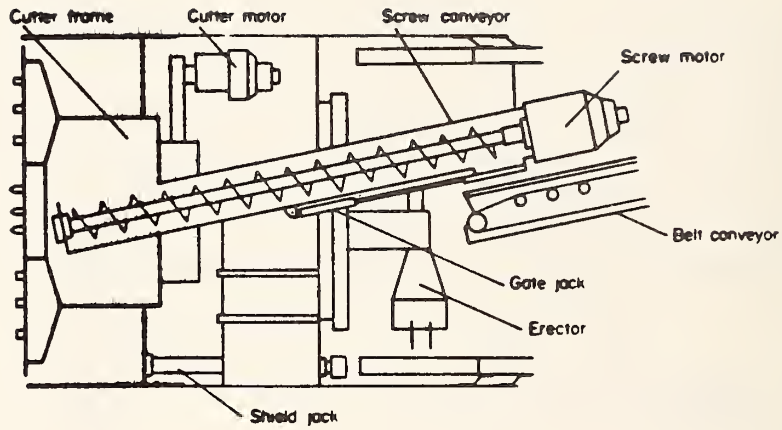
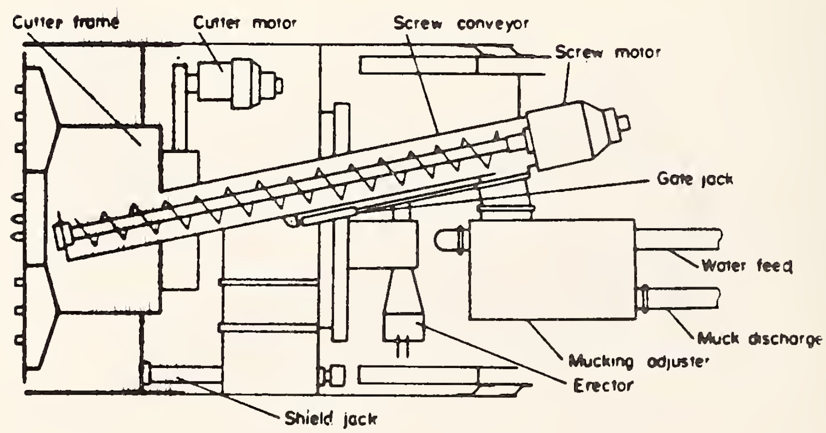


FIGURE 36 - Schematic of Slurry Shield (Ref. 201)

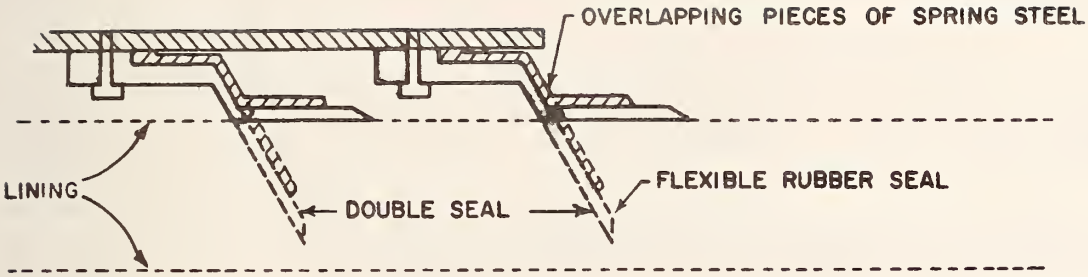


a) SOIL PRESSURE TYPE

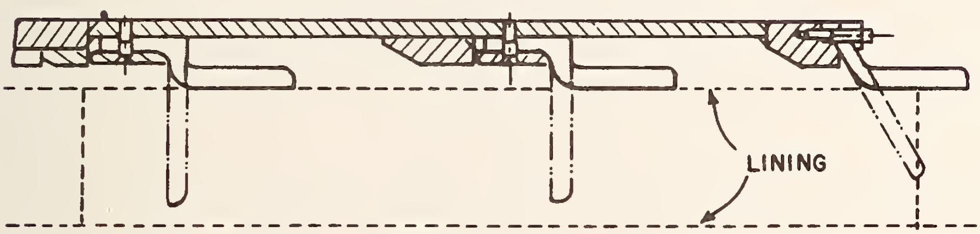


b) WATER PRESSURE TYPE

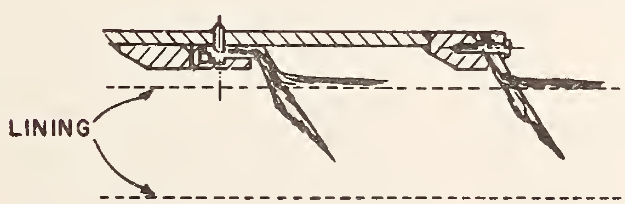
FIGURE 37 - Schematics of Earth Pressure Balance Shields (Ref. 142)



SPRING STEEL AND RUBBER



W-TYPE TAIL PACKING



DOUBLE WIRE BRUSH TAIL PACKING

FIGURE 38 - Japanese Tail Seals (Ref. 281)

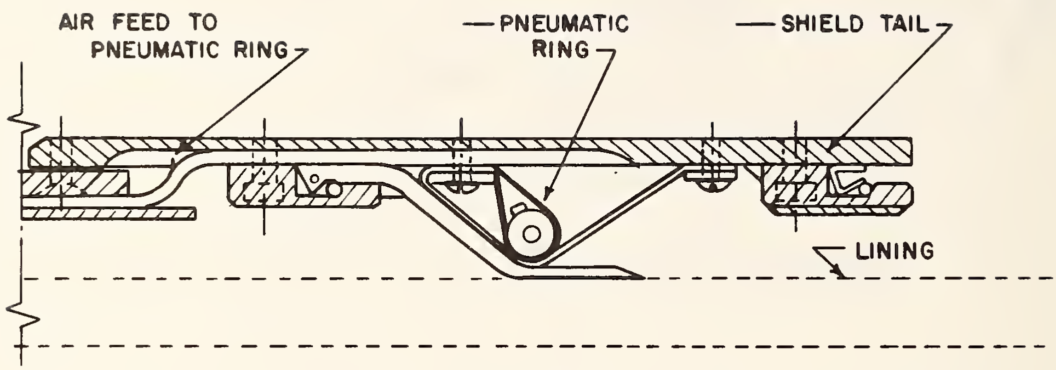


FIGURE 39 - A British Tail Seal (Ref. 50)

4.72 Construction

Successful use and application of a specialty shield is more related to an understanding of its behavior and use under a given set of conditions than details of the shield design.

Both slurry and EPB shields are highly specialized pieces of construction equipment that require careful attention to details of construction and maintenance for successful application. Because of their relatively complex design, they are relatively inflexible with regard to changes in subsurface conditions and therefore thorough subsurface investigation and careful planning of construction procedures is perhaps more important with this equipment than with other less complicated and more flexible methods.

Some key construction and maintenance considerations include:

1. Slurry shields require a substantial surface work site to accommodate the slurry cleanup equipment. In urban areas this can present problems because of space restrictions. Some contractors have attempted to reduce the size of required surface area by installing protions of the equipment in the shaft itself.

2. EPB shields handle muck disposal using conventional equipment. There is no recirculated slurry and therefore no large treatment facility is required at the surface.

3. Techniques for launching both types of shields are similar to those for compressed air. It is usually necessary to employ some alternate groundwater control method such as grouting, freezing or pre-drainage to permit launching of the shield. The ground must be stabilized while the shield is being readied for normal operation.

4. Intermediate shafts may be required at a closer spacing for slurry shields than EPB shields or other conventional shields because of slurry pumping considerations. Although the distance between tunneling machine and surface treatment plant can theoretically be extended an unlimited distance by adding pumps, it is normal practice to move the treatment facility to new working sites every 1 to 2 miles (1.6 to 3.2km) of tunnel advance.

5. Slurry and EPB shields work most efficiently in sand, silt and clays of low plasticity. Cobbles and boulders greater than 20 to 24 inches (50 - 60cm) present problems requiring special crushing and separation equipment.

Highly plastic clays create clogging problems and difficulty of separation from slurry. Figure 40 presents soil gradation guidelines for efficient shield operation. Different excavating heads are required for different ground, e.g., open face with picks for stiff clay, closed face with small openings and picks for sand and gravel and plated face with grillages, picks and discs for boulders. If changed conditions are encountered between shafts, it is extremely difficult to change machine excavating heads. It is a relatively simple operation to change the head at intermediate shafts.

6. Pervious sands and gravels can result in substantial loss of slurry.

7. The tail seal between the shield and tunnel liner has presented major construction problems due to slurry leakage into the working area. This problem has largely been solved through use of several types of seals including wire brushes and comibinations of rubber and spring steel as previously illustrated in Figures 38 and 39.

8. Whenever possible, it is preferable to use a native slurry due to the high cost of bentonite as well as potential environmental difficulties with disposal of used slurry.

9. Use of slurry and EPB shields eliminate the difficulties and reduced productivity associated with compressed air tunneling. Both types of shields work at deeper and shallower depths than is normally possible with compressed air. Shields can operate with as little as 3 to 7 ft (1 - 2m) of cover and at pressures up to 35 psi (242 kPa.). The effectiveness of tail seals is a limiting factor on the maximum head which can be withstood.

10. It is reported that surface settlement associated with slurry and EPB shields is typically one half of that associated with other tunneling. The British report (Ref. 50) that the volume of the settlement trough can be kept within the range of 0.5 to 1.4 percent of the total excavated cross-section.

11. A more accurate alignment can be maintained with slurry shields than is possible with other mechanical shields because the slurry in the heading reduces soil drag on the shield.

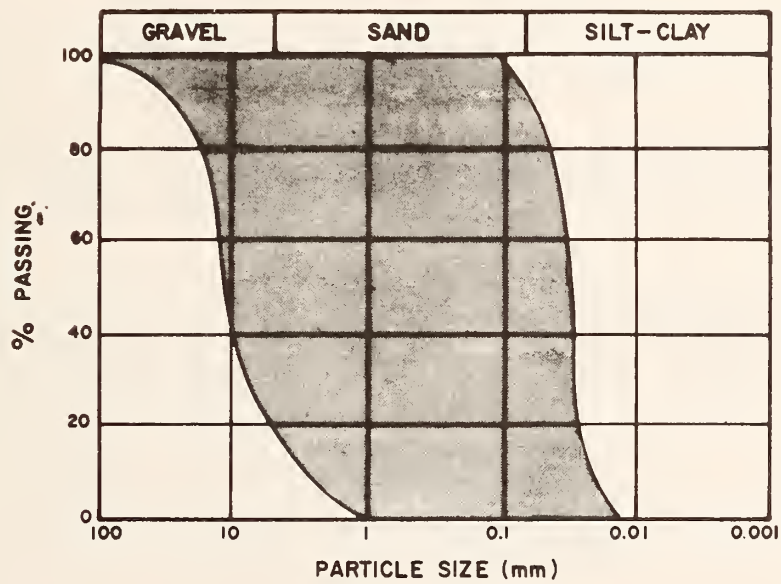


FIGURE 40 - Soil Gradation Guidelines for Efficient Operation of Slurry and Earth Pressure Balance Shields (Ref. 146)

4.80 ELECTRO-OSMOSIS

4.81 Design

Numerous studies have revealed that electro-osmotic transport can be related to water content, electrolyte concentration and soil minerology. Figure 41a is a useful summary which illustrates electro-osmotic flow versus water content for several clay-water-electrolyte systems. Electro-osmotic flow can be seen to vary over a little more than two orders of magnitude.

Key factors in the design of an electro-osmosis system include:

1. Quantity of water to be removed
2. Available time
3. Quantity of electrical power
4. Electrode type and spacing

The quantity of water removed at the cathode can be estimated for a one dimensional case by;

$$Q = (k_e)\left(\frac{\Delta E}{\Delta L}\right)\left(A\right)$$

where:

Q = Drainage ratio in cubic centimeters per second

k_e = electro-osmatic permeability in centimeters per second per volt per centimeter

ΔE = applied voltage in volts

ΔL = electrode spacing in centimeter

A = cross sectional area in square centimeters

For two or three dimensional flow patterns, this equation must be modified by shape factors derived from flow nets.

A good first approximation for electro-osmotic permeability, k_e, is 5×10^{-5} cm per sec. per volt per cm. although it can vary significantly from this value as illustrated in Figure 41b.

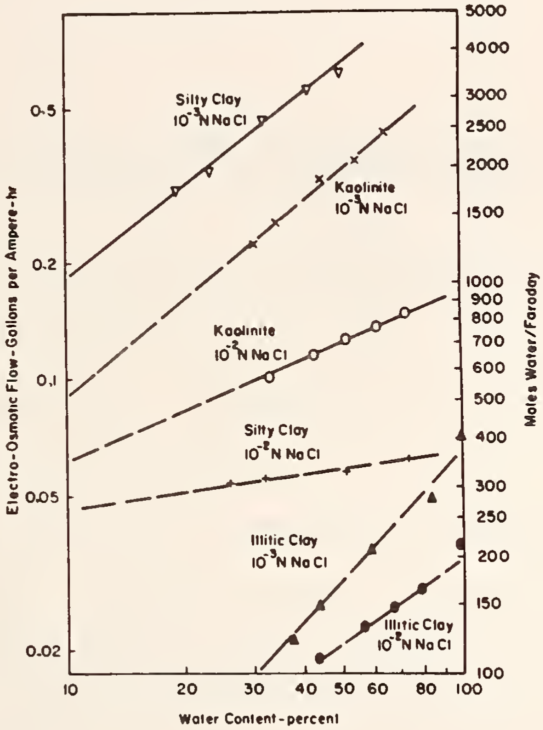


Figure 41a - Electro-Osmotic Flow Versus Water Content
in Clay-Water-Electrolyte Systems (Ref. 112)

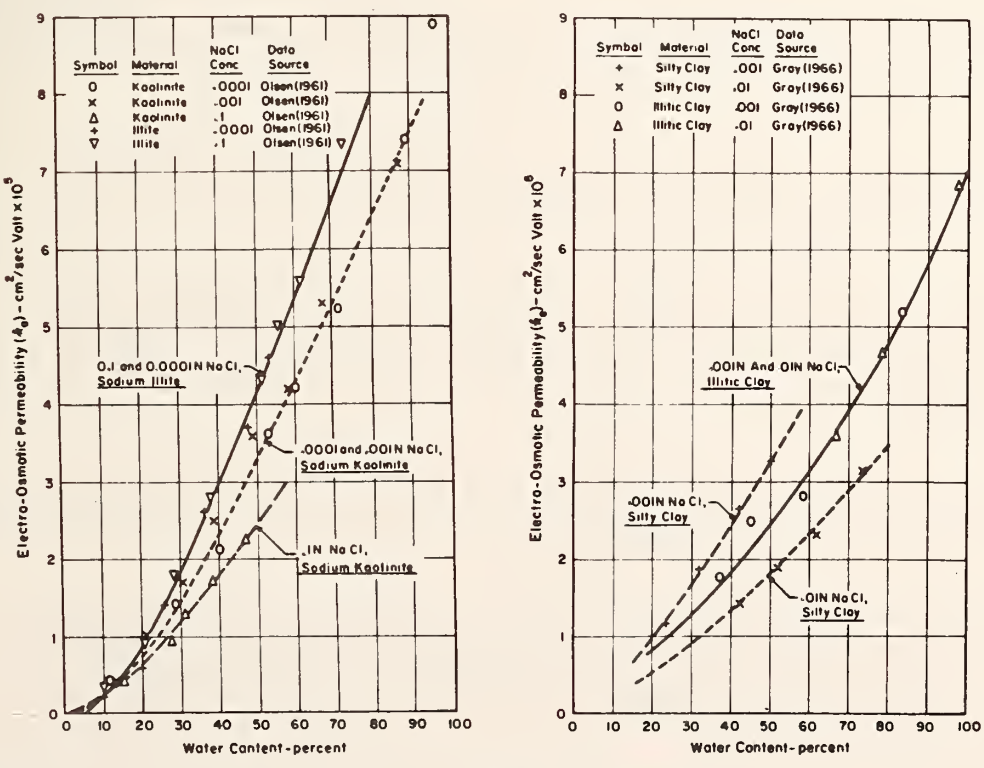


Figure 41b - Variations in Electro-Osmotic Permeability with Water Content
and Electrolyte Content and Electrolyte Content for Illitic and
Silty Clay (Ref. 112)

The electrical energy requirements per gallon discharged can be estimated from;

$$\epsilon = \frac{\Delta E}{k_i} \times 10^{-3}$$

where:

ϵ = electrical power in kilowatt hours

ΔE = voltage

k_i = electro-osmotic flow in gallons per ampere-hour

Using the above expressions one can perform an estimate of the economic and technical feasibility of electro-osmosis. The values calculated using Figure 41 and expressions above represent an upper limit of flows to be expected under field conditions because of flow interference due to gas generation at the electrodes, consolidation and fissuring of the soil, electrode decomposition and other retardation effects.

4.82 Construction

Once a tentative design has been developed, it is recommended that full scale test be conducted to verify anticipated electrical power consumption, electrode spacing and quantities of water removed. Some construction considerations include:

1. Portable DC power supplies such as welding machines are commonly used with applied voltages on the order of 100 volts.

2. Typical electrode spacings are on the order of 6 to 10 feet (1.8 to 3.0m). Wellpoints are commonly used as cathodes to facilitate water extraction. Simple concrete reinforcing steel may also be used as on electrode.

3. Typical energy consumption for past case histories is reported to vary from 0.5 kw hr per cu. of stabilized soil to 17.0 kwhr per cu. yd.

4. Electrode corrosion is to be anticipated and can be estimated using Faraday's laws. Electrode corrossion of 2.25 kg per cu. m. of treated soil is reported by Mitchell (Ref 186).

4.90 EXAMPLES OF GROUNDWATER CONTROL DURING CONSTRUCTION

This section contains a series of eleven examples illustrating technically feasible methods for groundwater control during construction. Five cut-and-cover and six bored tunnel examples are presented. They include:

Cut-and-Cover Tunnels

1. Uniform granular soils
2. Uniform granular soils overlain by organic silt and fill
3. Stratified silt and sand
4. Clayey silt underlain by gravelly sand below invert
5. Stratified silt and sand and fractured rock

Bored Tunnels

6. Uniform granular soils
7. Uniform granular soils overlain by organic silt and fill
8. Stratified silt and sand
9. Clayey silt underlain by gravelly sand
10. Stratifed silt and sand and fractured rock
11. Fractured rock

A description of each follows:

4.90.1 EXAMPLE 1 - CUT-AND-COVER TUNNEL IN UNIFORM GRANULAR SOIL

A. Project Conditions; (Figure 42)

- Cut-and-Cover Tunnel, Urban Area
- Depth of excavation = 40 feet (12.2m)
- Soils: 10 feet (3.0m) Miscellaneous FILL
  110 feet (33.5m) Medium dense, fine to coarse, gravelly SAND
- Depth to groundwater = 10 feet (3.0m)

B. Feasible Methods (Figure 4)

Primary	Secondary
1. Wells	1. Grouting
2. Wellpoints	2. Partial Cut-Off
3. Full Cut-Off	3. Freezing

C. Comments

1. Dewatering:
   - Required drawdown = 30^{+} feet (9.1m)
   - Relatively pervious soils, therefore large volumes to be pumped
   - Probably no effect on adjacent structures (dense granular soils)

   - Wellpoints - Lift limitations require multi stages which may clutter the excavation.
   - Ejectors - Inefficient when large volumes must be pumped
   - Wells - The best way to handle large volumes of water with widely spaced installations

2. Cufoffs: - Positive waterproofing, cutoff should intercept deep impervious stratum. Probably not economically justified in this situation.

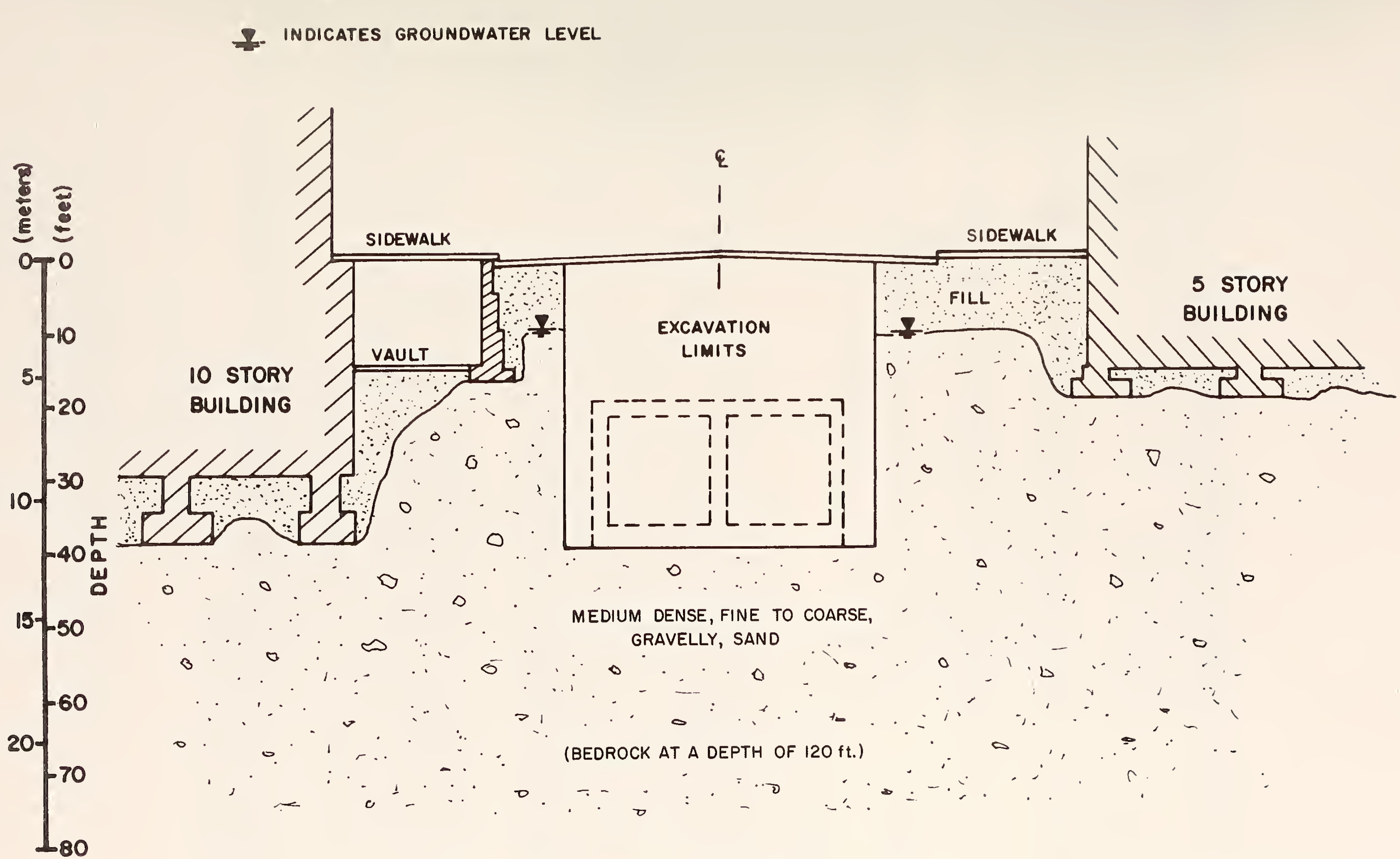


FIGURE 42 - Example 1, Cross Section

- For partial waterproofing, cutoff must extend deep enough to reduce piping and heave into excavation base; Secondary sumping required in excavation

3. Grouting:
   - Cement grout can be used if soil matrix is coarse. Silicate grouts feasible in finer portions.
   - Grouting may be erratic if soil is stratified with layers of varying permeability

4. Freezing:
   - Expensive, but technically feasible. Complete cut-off if bottom frozen. Hydrostatic uplift must be considered

5. Summary Discussions

In this relatively simple soil profile the usual method chosen would be dewatering by deep wells. The ability to handle large quantities of water from a few, relatively widely spaced locations usually gives wells an economic advantage over other dewatering methods. Special considerations such as union restrictions on the number of pumps per operator or environmental limitations on the allowable quantity of water to be pumped may lessen this economic advantage to where a lesser rated method may become economically competitive. Recharge is probably unnecessary except for special environmental considerations.

The use of any of the cutoff methods is possible if the soil does not contain boulders or other obstructions such as utility lines. Diaphragm walls or sheeting can also act as a lateral support system for the excavation. The cutoff methods would be more attractive if an impervious stratum existed at a shallower depth.

Both grouting and freezing techniques could provide positive waterproofing for this type of excavation. The final choice may depend primarily on relative costs and local experience.

4.90.2 EXAMPLE 2 - CUT-AND-COVER TUNNEL IN UNIFORM GRANULAR SOIL OVERLAIN BY ORGANIC SILT

A. Project Conditions (Figure 43)

- Cut-and-Cover Tunnel, Urban Area
- Depth of Excavation = 40 feet (12.2m)
- Soils: 10 feet (3.0m) Miscellaneous Fill
  10 feet (3.0m) Soft, Organic SILT
  100 feet (30.0m) Medium dense, fine to coarse, gravelly SAND, overlying bedrock
- Depth to Groundwater = 10 feet (3.0m)

B. Feasible Methods (Figure 4)

Primary	Secondary
1. Wells	1. Grouting
2. Wellpoints	2. Partial Cut-Off
3. Full Cut-Off	3. Freezing

C. Comments

1. Dewatering: - Required drawdown = 30^{+} feet (9.1m)

   - Relatively pervious soils, therefore large volumes to be pumped
   - Adjacent structures founded on granular material however, utility lines, roads, and sidewalk vaults may be affected by some settlement due to compression of organic silt.
   - Recharge may be required to minimize consolidation of organic silt.

   - Wellpoints - Lift limitations require multi stage system which may clutter the excavation.
   - Ejectors - Are inefficient when large volumes are to be pumped.
   - Wells - The best way to handle the large volume of with widely spaced installation.

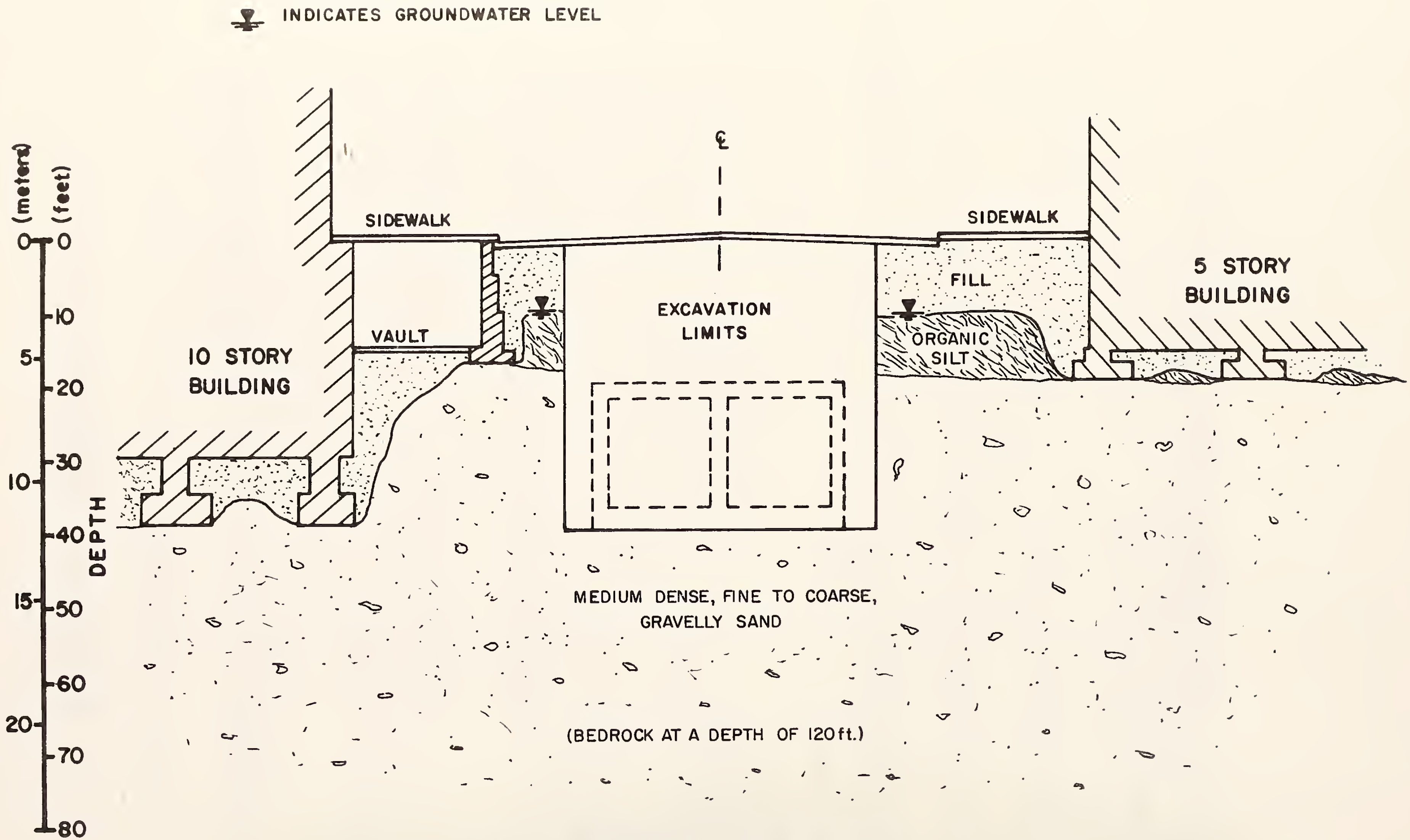


FIGURE 43 - Example 2, Cross Section

2. Full Cut-Off

   - See Example 1

3. Grouting

   - See Example 1

   - Grout penetration of organic silt very unlikely even though a very low viscosity grout.

4. Freezing

   - Could effectively stabilize the entire stratum; heave may be a factor in the silt layer

5. <u>Summary Discussions</u>

The advantages of wells are essentially the same as indicated in the previous example. Possible compression of the organic silt is a special concern. A shallow recharge system may be required as a contingency. However, the economics must be weighed in repairing damages due to settlement of these shallow installations versus the cost of recharge to prevent consolidation.

A cutoff could be feasible if it could be installed without encountering obstructions or boulders during driving (steel sheeting) or trench excavation (slurry walls, slurry trenches). The cutoff would be even more desirable if an impervious stratum existed at a shallower depth. Sheeting or slurry walls can also act as components of the lateral support systems.

Grouting is probably not feasible for stabilizing the entire profile and so would have to be used in conjunction with another waterproofing technique in the silt such as a cut-off. Freezing is possible, though probably more expensive than other methods. Possible heave of the organic silt could cause damage to adjacent structures and utilities and should be monitored.

4.90.3 EXAMPLE 3 - CUT-AND-COVER TUNNEL IN STRATIFIED SILT AND SAND

A. Project Conditions: (Figure 44)

- Cut-and-Cover Tunnel, Urban Area
- Depth of Excavation = 40 feet (12.2m)
- Soils: 10 feet (3.0m) Miscellaneous FILL
  110 feet (33.5m) Medium dense, stratified, SAND, silty SAND and SILT overlying bedrock
- Depth to groundwater = 10 feet (3.0 m)

B. Feasible Methods (Figure 4)

Primary

1. Wellpoints or ejectors if sand overlies silt
2. Wells if silt overlies sand
3. Full Cut-Off

Secondary

1. Grouting if relatively sandy
2. Partial Cut-Off
3. Freezing

C. Comments

1. Dewatering:
   - Required drawdown = 30^{+} feet (9.1m)
   - Depending on the permeability and frequency of sand strata, large volumes of water may be pumped
   - Recharge is probably not required, since compressible soils are not indicated

   • Wellpoints - Lift limitations require a multi-stage system which may clutter excavation; however, vacuum may have a tremendous affect in stabilizing the finer grained strata

   • Ejectors - If large pumping volumes are not required, this method has the advantage of creating vacuum in the soils as do wellpoints; however, only a single stage installation is required.

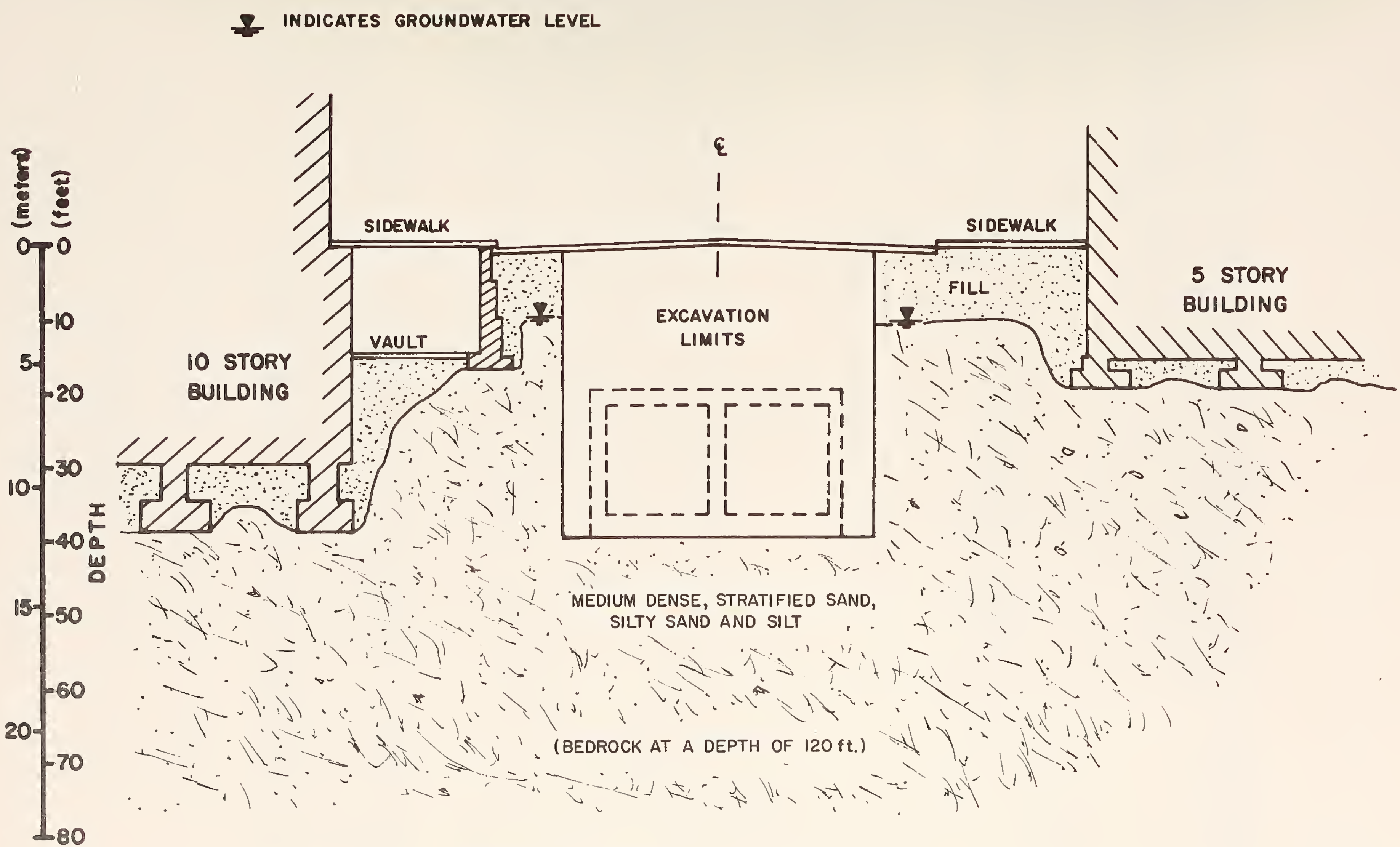


FIGURE 44 - Example 3, Cross Section

- Wells - Highly stratified soils are difficult to drain by gravity. Depending on the required pumping volume, wells become more competitive at greater spacings. Vacuum may be applied to the well to enhance the drainage.

2. Cutoffs: - See Example 1

3. Grouting: - Probably not very effective if the soil is highly stratified. A low viscosity grout would be required to insure proper penetration into all strata.

4. Freezing: - An effective alternative for the stabilization of the entire profile. Heave may occur in low permeability silt layers

5. Summary Discussions

Depending on the degree of stratification and relative permeabilities of the different layers, a number of dewatering methods may be appliable. Predrainage may be difficult if the low permeability strata are relatively continuous. This would retard the vertical drainage and possibly cause a number of perched water conditions to exist. The creation of a vacuum in the soils usually has a tremendous effect in stabilizing fine sands and silts. Ejectors, having this capability would be the preferred dewatering method. However, if large volumes must be handled, vacuum enhanced deep wells may be competitive.

All cutoff and freezing techniques are well suited to this type of soil profile if they can be constucted deep enough to effectively reduce flows and resulting gradients into the excavation bottom. Grouting is possible if the grout is of a viscosity low enough to permeate the finest grained soil in the profile. This requirement may necessitate an expensive resin grout.

4.90.4 EXAMPLE 4 - CUT-AND-COVER TUNNEL IN CLAYEY SILT UNDERLAIN BY GRAVELLY SAND BELOW BOTTOM OF EXCAVATION

A. Project Conditions (Figure 45)

- Cut-and-Cover Tunnel, Urban Area
- Depth of Excavation - 40 feet (12.2m)
- Soils: 10 feet (3.0m) Miscellaneous FILL
  35 feet (10.7m) Medium stiff clayey SILT
  25 feet (7.6m) Medium dense, fine to coarse gravelly SAND
  50 feet (15.2m) Stiff CLAY
- Depth to Goundwater - 10 feet (3.0m)

B. Feasible Methods (Figures 4 and 6)

Primary	Secondary
1. Wells	1. Freezing
2. Wellpoints*	2. Partial Cut-Off
3. Ejectors*	
4. Full Cut-Off	

* If drainage required after pressure relief.

C. Comments

1. Dewatering - Require pressure relief at excavation base = 30^{+} feet (9.1m)

   - Depending on the amount of cohesion of the silt, predrainage may not be necessary, or may be handled using only sumping after pressure relief.

   - If the silt has little to no cohesion, predrainage may be required to prevent running or sloughing due to residual seepage.

   - If the silt is very clayey and therefore compressible, recharge may be required.

   • Wellpoints - Not applicable to pressure relief. Multistage system required if silt must be predrained.

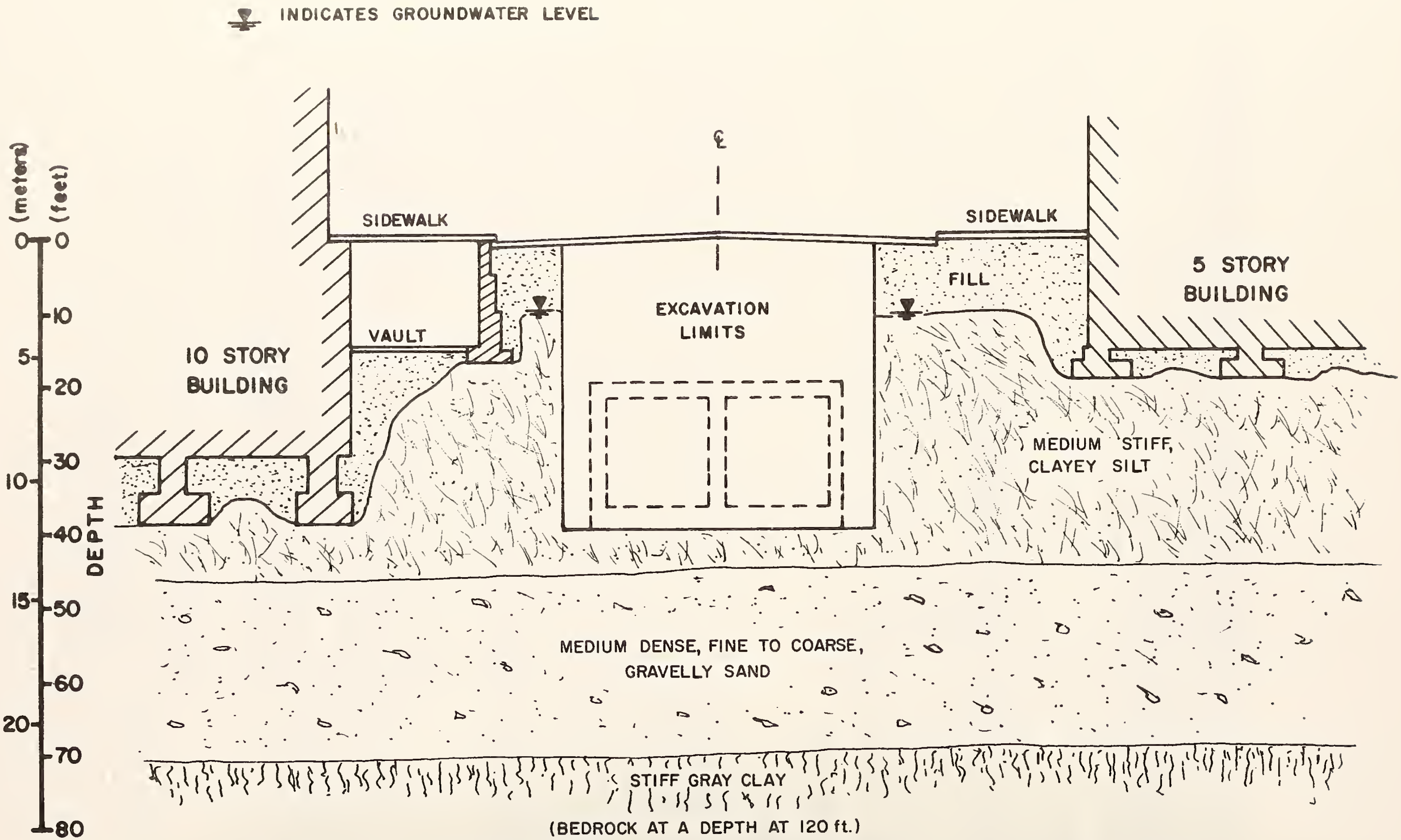


FIGURE 45 - Example 4, Cross Section

- Ejectors - Volume for pressure relief makes method inefficient. May be helpful in stabilizing a noncohesive runny silt.

- Wells - Most effective tool for pressure relief, probably ineffective in stabilizing silt.

2. Cutoffs

   - See Example 1

   - If carried through the underlying gravelly sand, it could be an effective protection against bottom blow.

3. Freezing

   - Can be used to create vertical impermeable cutoff down to an impervious layer, or to create a frozen zone beneath the base of the excavation only.

4. Grouting

   - Not applicable in clayey silt

   - May be used to stabilize base of excavation against bottom blow

5. Electro-Osmosis

   - Can effectively stabilize silt stratum during construction if it is fine enough.

6. <u>Summary Discussions</u>

   This profile provides a number of problems with respect to dewatering due to the large differences in permeabilities and compressibilities of the two strata. Gravity drainage of the clayey silt would be very slow and difficult and could result in intolerable settlements. The highly pervious sand and gravel stratum is a potential source for large flows into the base of the excavation and thus requires pressure relief to prevent bottom blows.

   Deep wells would normally be used to reduce hydrostatic pressure on the subgrade of the excavation. Depending on the permeability and plasticity of the silt, predrainage of the upper soils may not be required. If predrainage of the silt is required, ejectors would be a suitable

method. Recharge to prevent settlements outside the excavation may be required, but tight cutoffs or underpinning may be easier and more economical.

Structural cutoff walls or vertical freeze walls could intercept a deep impervious stratum to reduce underseepage. Steel sheeting has an inherent permeability near that of the silt and so would not be very effective in maintaining water levels outside of the excavation. In either case, the cutoff would have to be watertight to resist the high gradients which would develop as the excavation was dewatered.

Grouting or freezing of the excavation base could provide a partical cutoff from the sand layer. Other methds would then be required to stabilize the overlying silt.

4.90.5 EXAMPLE 5 - CUT-AND-COVER TUNNEL IN STRATIFIED SILT AND SAND AND FRACTURED BEDROCK

A. Project Conditions (Figure 46)

- Cut-and-Cover Tunnel, Urban Area
- Depth of Excavation = 40 feet (12.2m)
- Soil and Rock: 10 feet (3.0m) Miscellaneous FILL
  20 feet (6.1m) Stratifed SILT AND SAND
  overlying fractured BEDROCK which is
  more pervious than the overlying soils.
- Depths to groundwater = 10 feet (3.0m)

B. Feasible Methods (Figure 4)

Primary	Secondary
1. Wells	1. Grouting
2. Wellpoints	2. Freezing
	3. Partial Cut-Off

C. Comments

1. Dewatering: - Required drawdown = 30 feet (9.1m)
   - Fractured rock could supply a large volume of water
   - Stratified upper soils may be difficult to predrain by gravity
   - Possibly separate systems to dewater soil and rock

   • Wellpoints - Multiple stages required to predrain to top of rock. Very expensiveto drill closely spaced holes in rock

   • Ejectors - A single stage could predrain to top of rock depending on the volume required to be pumped. The ability to create a vacuum may stabilize silt layers. Very expensive to drill closely spaced holes in rock

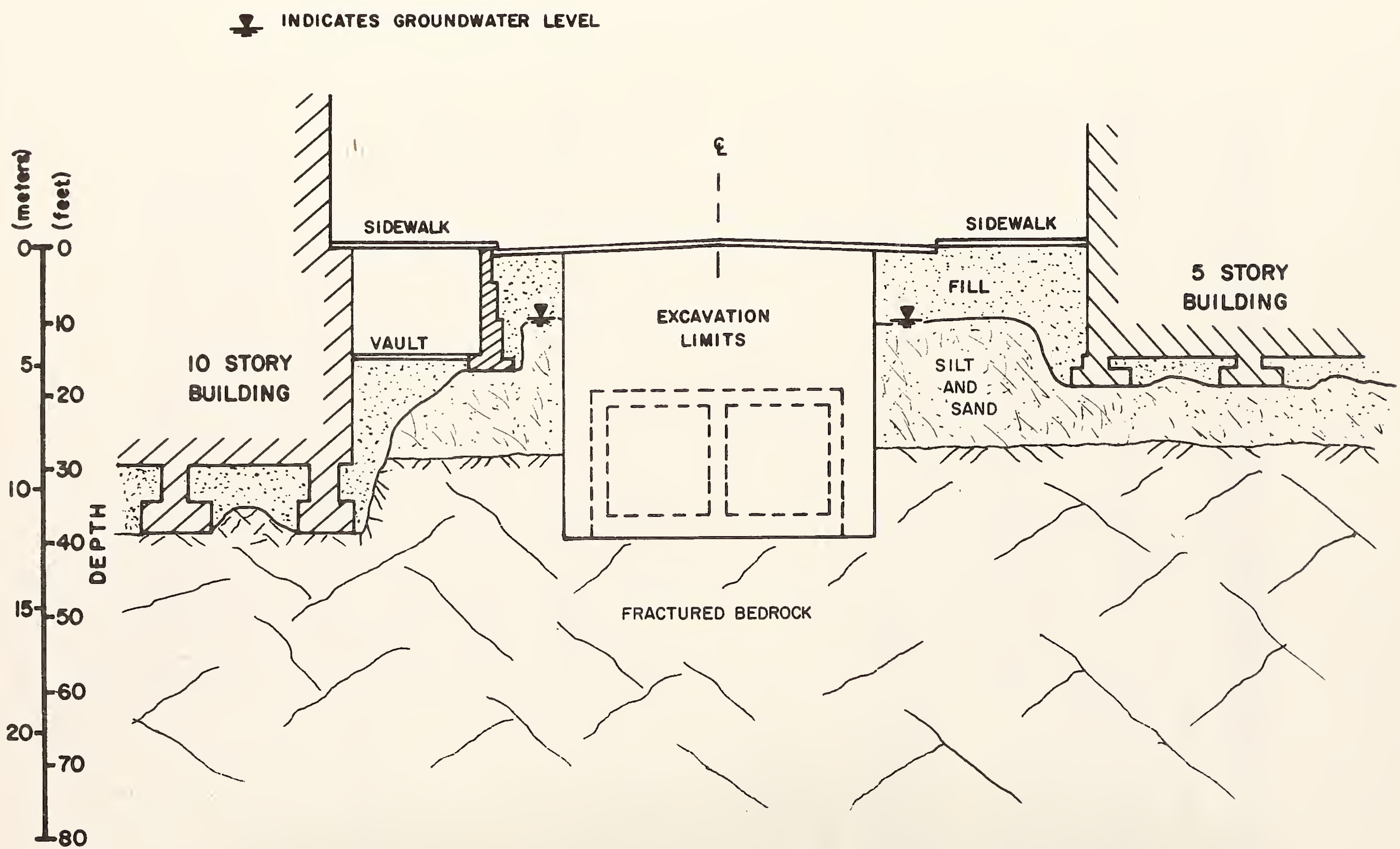


FIGURE 46 - Example 5, Cross Section

- Wells - Depending on amount of water to be pumped from rock, wells would be the mostly likely method. If flow from rock is small, then open sumping may be sufficient

3. Grouting: - Very difficult to insure adequate penetration into stratified soils

   - Coarse particulate grout required to seal off large seams in the rock

4. Freezing: - Possible, but likely to be expensive because of drilling costs in rock

5. Summary Discussions

Two separate groundwater conditions must be addressed. The soils above rock require predrainage unless they can be cutoff at the rock surface. Any irregular rock surface could produce openings which must be sealed to prevent loss of ground. Residual seepage along the rock surface will probably require local sumping. Depending on the permeabilities of the sands and continuity of the low permeability strata, predrainage may be accomplished with either ejectors or wells. The rock may produce large groundwater flows. The rock precludes most cutoff methods due to the inherent difficulty of penetrating far enough to intercept sound rock and stop seepage. Grouting of the rock is possible, yet difficult if it is highly decomposed. Grouting of the overlying sands and silts may be feasible with an low viscosity chemical grouts.

Dewatering using deep wells appears to be the only reasonable approach to handle the large volumes of water and large lift capacities which would undoubltedly be necessary. Freezing the entire excavation is feasible, but probably much more expensive than dewatering methods.

4.90.6 EXAMPLE 6 - BORED TUNNEL IN UNIFORM GRANUALR SOIL

A. Project Conditions (Figure 47)

- 20 foot (6.1m) Diameter Bored Tunnel, Urban Area
- Depth of invert = 70 feet (21.3m)
- Soils: 10 feet (3.0m) Miscellaneous fill overlying bedrock at a depth of 120 feet (36.6m)
  110 feet (33.5m) Medium dense, fine to coarse, gravelly SAND
- Depth of groundwater - 10 feet (3.0m)

B. Feasible Methods (Figure 5)

Primary	Secondary
1. Wells	1. Grouting
2. Full-Cut-Off	2. Freezing
	3. Slurry on EPB Shields, Compressed Air
	4. Partial Cut-Off

C. Comments

1. Dewatering:
   - Required drawdown = 60^{+} feet(18.3m).
   - Pervious soils, therefore large volumes to be pumped.
   - Recharge not required unless for special considerations.

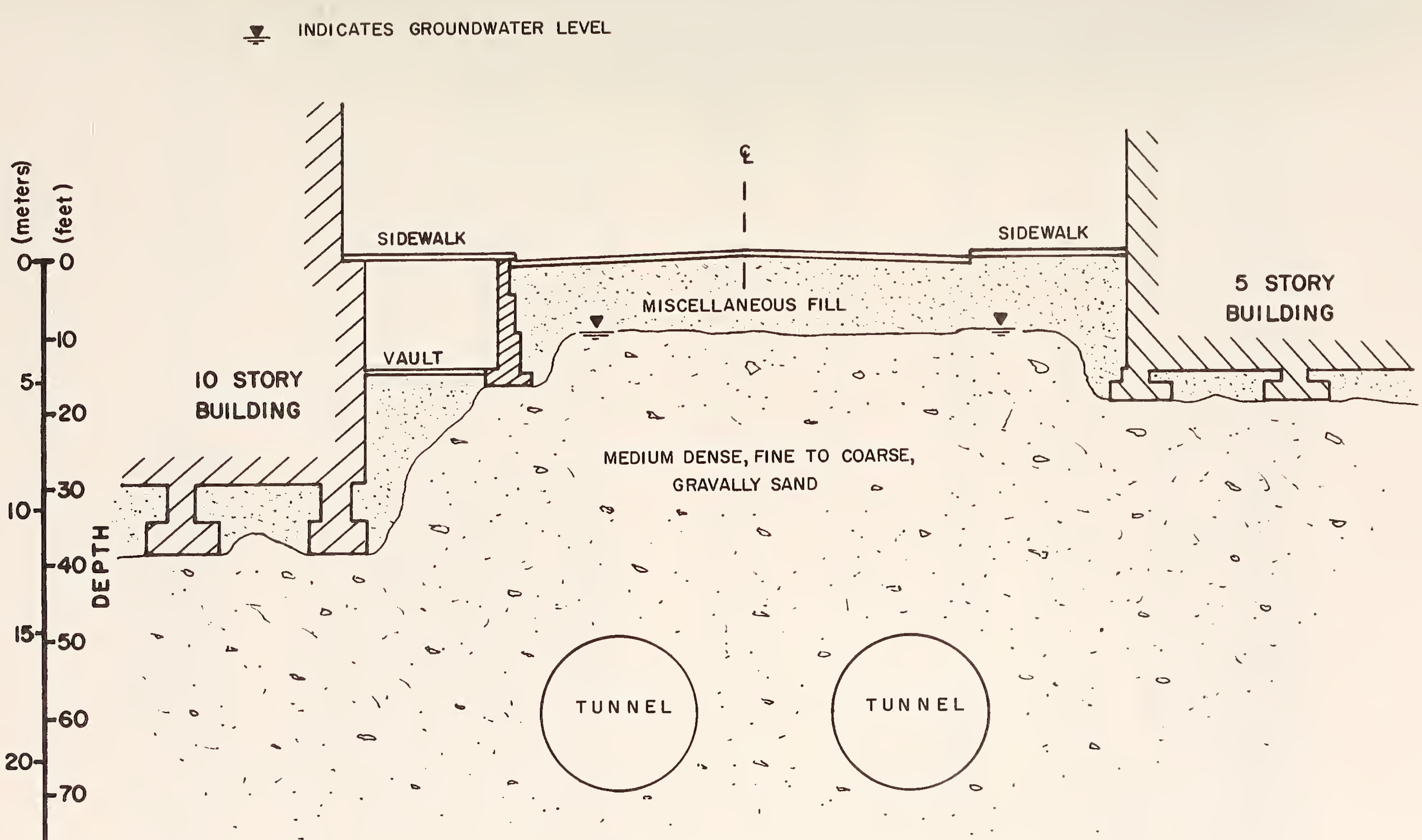
   • Wellpoints - Not applicable in bored tunnels except in special problem cases.

   • Ejectors - Not efficient due to large required pumping volume.

   • Wells - The usual method for groundwater control in uniform relatively permeable soils.

2. Cutoffs

   - Typically not used in bored tunnel construction, though slurry trench cutoffs or thin wall cutoffs may prove economical



(BEDROCK AT A DEPTH OF 120 ft.)

FIGURE 47 - Example 6, Cross Section

- Cutoff should penetrate to an impervious layer for positive dewatering; partial penetration combined with sumping inside excavation may also be adequate.

3. Grouting

   - Inexpensive cement grout may be used if sand stratum is coarse.

   - May not be effective if sand is highly stratified.

4. Freezing

   - Freezing can be conducted from surface or at tunnel level to create impervious frozen zone

5. Compressed Air/Slurry/EPB Shields

   - Large air or slurry losses probable in coarse granular soils

   - High air pressures would severely limit working times.

   - Slurry and EPB Shields have difficulty handling cobbles.

6. Summary Discussions

   Traditionally, predrainage or limited predrainage in conjunction with compressed air has been used. Deep wells have been the usual tool for groundwater control in uniform pervious soils. Large quantities of water can thus be handled from relatively widely spaced locations. Assuming the primary lining and waterproofing follow close behind the tunnel heading, wells can be systematically turned on in advance of the heading and shut off after the waterproofing and lining is placed. Therefore, restrictions on union operation of a specified number of pumps is less likely to become a problem. If compressed air use is anticipated, partial lowering of the water level with deep wells can reduce the required air pressure in the tunnel.

   Grouting or freezing to create an impervious zone surrounding the tunnel are also possible solutions. Cutoffs see limited use in bored tunnel schemes. They have traditionally been used an combined dewatering/support systems in cut and cover construction.

Slurry shields or EPB Shields are well suited for granular water bearing strata as long as numerous cobbles or boulders are not present. Just as air losses may pose a problem with compressed air, slurry and water losses may be a concern in highly pervious soils.

4.90.7 EXAMPLE 7 - BORED TUNNEL IN UNIFORM GRANULAR SOIL OVERLAIN BY ORGANIC SILT

A. Project Conditions (Figure 48)

- 20 feet (6.1m) Diameter Bored Tunnel, Urban Area

- Depth to Invert = 70 feet (21.3m)

- Soils: 10 feet (3.0m) Miscellaneous Fill
  10 feet (3.0m) Soft, Organic SILT
  100 feet (30.0m) Medium dense, fine to coarse, gravelly SAND, overlying bedrock

- Depth to Groundwater = 10 feet (3.0m)

B. Feasible Methods (Figure 5)

Primary	Secondary
1. Wells	1. Grouting
2. Full Cut-Off	2. Slurry or EPB Shields
	3. Ejectors
	4. Freezing
	5. Compressed Air

C. Comments

1. Dewatering: - Required drawdown = 60^{+} feet (18.3m)

   - Large volumes to be pumped in the pervious sand stratum

   - Recharge may be required because of possible compression of the organic silt due to groundwater lowering.

   - Wellpoints - Not applicable in bored tunnels

   - Ejectors - Volume to be pumped makes use of ejectors inefficient

   - Wells - Best tool to handle the large quantities to water to be pumped

2. Cutoffs: - See Example 6

3. Grouting - See Example 6

4. Freezing - See Example 6

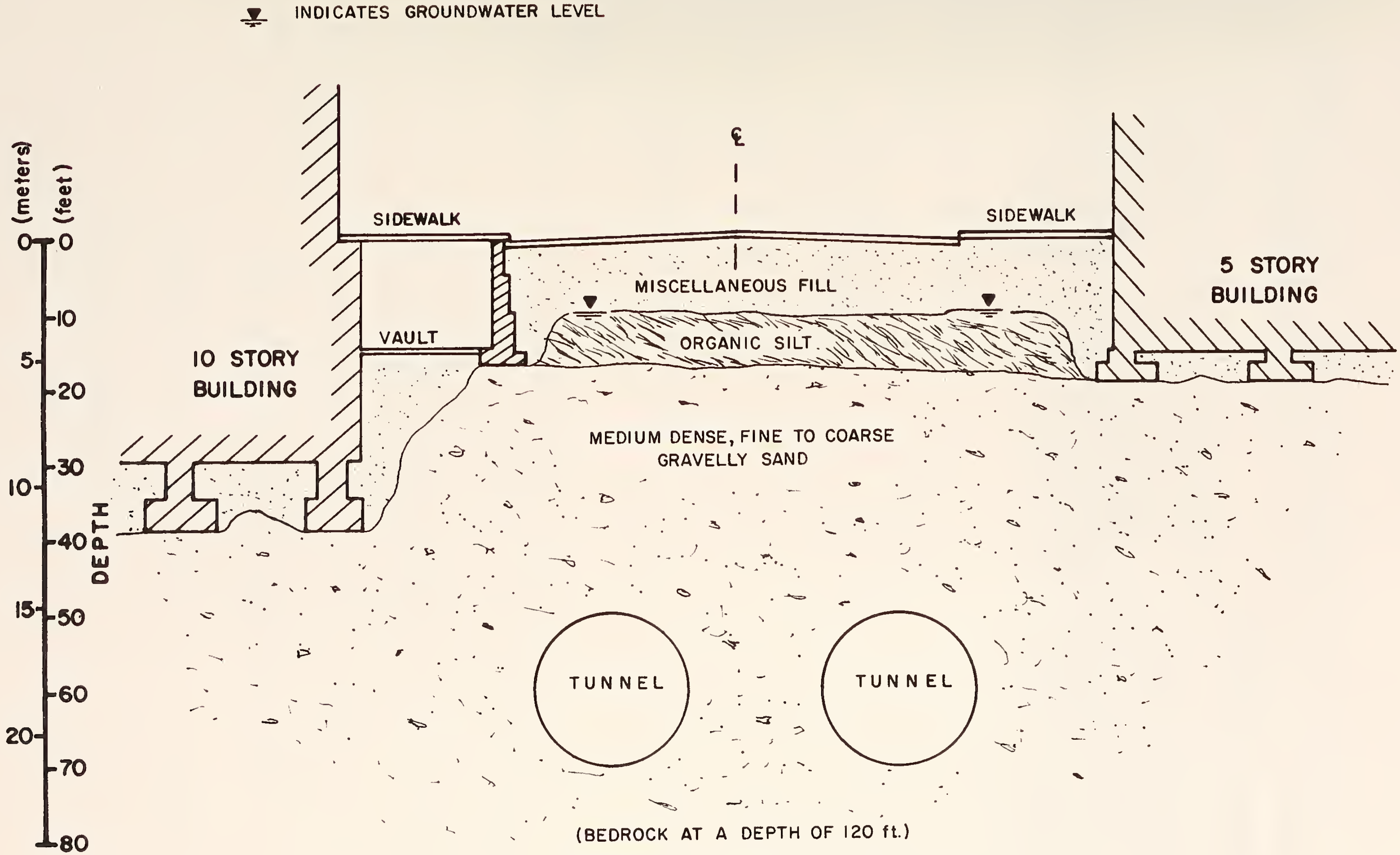


FIGURE 48 - Example 7, Cross Section

5. Compressed Air/Slurry/EPB Shields - See Example 6

6. Summary Discussions

This profile is very similar to the uniform granular profile of Example 6. The only new factor to consider is the compression of the overlying silt layer using a predrainage scheme. The resulting settlement may be prohibitive and require recharge to maintain groundwater levels in the silt for protection of utilities. Buildings bearing on granular soil should not be in danger of significant damage.

4.90.8 EXAMPLE 8 - BORED TUNNEL IN STRATIFED SILT AND SAND

A. Project Conditions (Figure 49)

- 20 feet (6.1m) Diameter Bored Tunnel, Urban Area
- Depth to Invert = 70 feet (21.3m)
- Soils: 10 feet (3.0m) Miscellaneous FILL
  110 feet (33.5m) Medium Dense, stratified SAND, silty SAND and SILT overlying bedrock
- Depth to Groundwater = 10 feet (3.0m)

B. Feasible Methods (Figure 5)

Primary	Secondary
1. Compressed Air, Slurry or Earth Pressure Balance Shields	1. Grouting
2. Wells if primarily sand	2. Freezing
3. Ejectors if primarily silt	

C. Comments

1. Dewatering:
   - Required drawdown = 60^{+} feet (18.3m)
   - Recharge probably not requried since soil is granular
   - Stratification may prohibit gravity drainage and result in perched conditions.

- Wellpoints - Not applicable in bored tunnels.
- Ejectors - May be used if volumes not too great. The effect of vacuum may enhance the stabilization of the silts.
- Wells - If large volumes are expected, i.e. primarily a sand.
  - Should be vacuum enhanced to aid stabilization of finer materials.

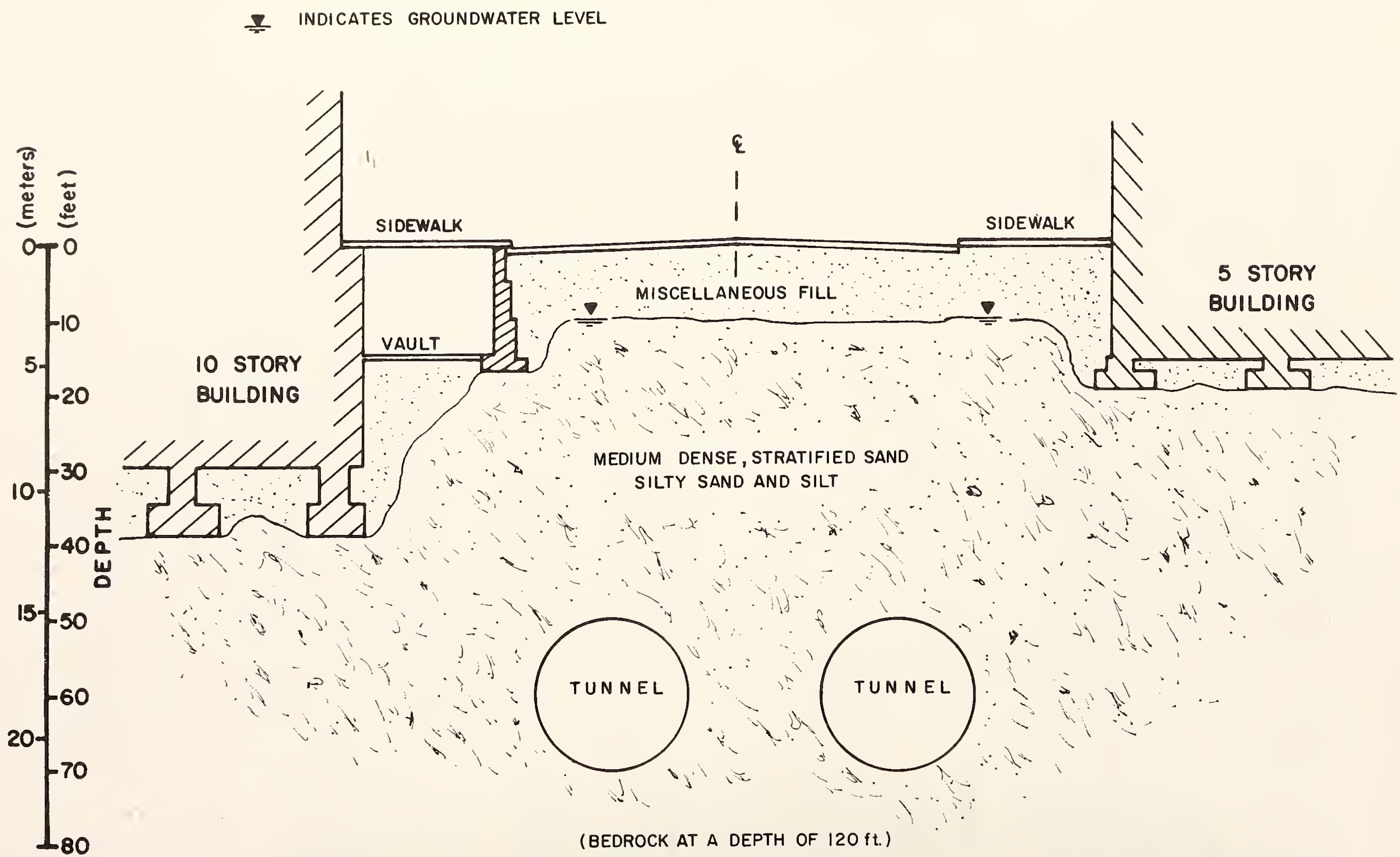


FIGURE 49 - Example 8, Cross Section

2. Cutoffs

   - See Example 6

3. Grouting

   - Probably not applicable in stratified soil, especially if silty. Low viscosity grouts would be required to insure proper penetration of all strata.

4. Compressed Air/Slurry/EPB Shields

   - Limited practical working pressures in compressed air shields. Possible combination with a dewatering method to reduce air pressures.

   - Very useful in silty granular soil due to small air and slurry losses

   - Limited use of slurry and EPB shields in the United States

5. Freezing

   - See Example 6. Some danger of frost heave in silty soils

D. Summary Discussions

Dewatering or dewatering in conjunction with compressed air are both well suited to this profile. Slurry or EPB shields are also applicable due to the nature of the existing sands and silts. Provisions should be made to reduce seepage into the tunnel if the sands and silts are susceptible to running or sloughing. Depending on the type of construction method chosen, minor flows of water at the heading may be disasterous. It is doubtful that complete drainage can be acheived by any dewatering method. Both freezing and cutoff methods are technically feasible but probably not economically competitive with the above mentioned techniques. Grouting in a finer grained stratified soil is very difficult to control and expensive due to the need for low viscosity chemical grouts.

4.90.9 EXAMPLE 9 - BORED TUNNEL IN CLAYEY SILT UNDERLAIN BY GRAVELLY SAND

A. Project Conditions (Figure 50)

- 20 feet (6.1m) Diameter Bored Tunnel, Urban Area
- Depth to Invert = 70 feet (21.3m)
- Soils: 10 feet (3.0m) Miscellaneous FILL

  60 feet (18.2m) Medium stiff, clayey SILT

  20 feet (6.1m) Medium dense, gravelly, medium to coarse SAND

  50 feet (15.2m) Stiff, silty CLAY
- Depth to Groundwater = 10 feet (3.0m)

B. Feasible Methods (Figures 5 and 6)

Primary

1. Wells
2. Compressed air, slurry or EPB shield
3. Full Cut-Off

Secondary

1. Freezing
2. Ejectors if groundwater control in silt required

C. Comments

1. Dewatering: - Pressure relief is required at invert

   - If the silt has low cohesion predrainage may be required. If piezometric head is lowered for enough time to initiate consolidation, recharge may have to be considered.

   • Wellpoints - Not applicable in bored tunnel

   • Ejectors - Inefficient for pressure relief due to high volume to be pumped. May be useful in stabilizing running silt.

   • Wells - Most effective tool for pressure relief. Probably inefficient in stabilizing silt.

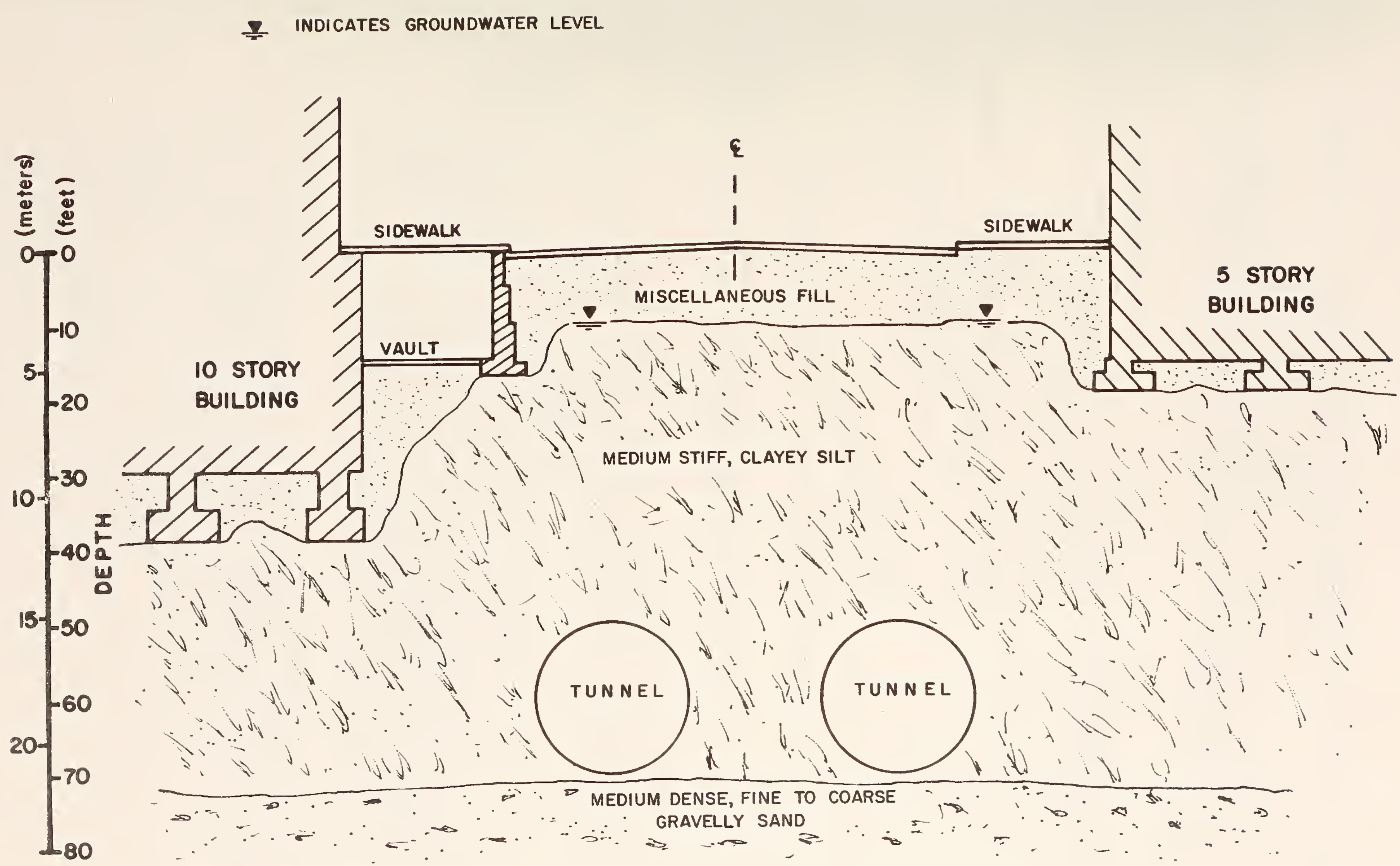


FIGURE 50 - Example 9, Cross Section

2. Cutoffs

- Not normally used in tunnels, but could be effective in cutting off recharge to gravelly sand below the tunnel invert.

3. Grouting

- See Example 4

4. Freezing

- Like cutoffs, can be used to create a vertical seepage barrier if keyed into an impervious stratum

- Horizontal freezing can create an impervious zone surrounding the tunnel

5. Compressed Air/Slurry/EPB Shields

- Well suited to low permeability soils

- Pressure relief may be required in the underlying gravels to prevent bottom instability

6. Summary Discussions

The main concern in this situation is pressure relief in the gravelly sand stratum which lies just beneath the tunnel invert. Since total predrainage of the entire profile would be very difficult due to the low permeabilty of the silt, a combined system of pressure relief along with a second groundwater control method may be required. Deep wells would normally be used to accomplish pressure relief. Compressed air or slurry shield would be the recommended tunneling method. Ejectors may be useful in reducing air pressure requirements, but may require considerable lead time to produce drainage. In such a case consolidation may be a problem.

Grouting is probably only possible in the sand and gravel layer to prevent bottom blows. Cutoffs or freezing are, of course, technically feasible but probably not economical compared to compressed air and/or dewatering. Similarly, electro-osmosis is capable of stabilizing the silt but may be an extreme measure not required except in unusual instances.

4.90.10 EXAMPLE 10 - BORED TUNNEL IN STRATIFIED SILT AND SAND OVERLYING FRACTURED ROCK

A. Project Conditions (Figure 51)

- 20 feet (6.1m) Diameter Bored Tunnel, Urban Area
- Depth to Invert = 20 feet (21.3m)
- Soil and Rock: 10 feet (3.0m) Miscellaneous FILL
  50 feet (15.2m) Medium dense silty SAND stratified with loose SILT overlying fractured dolomite.
- Depth to Groundwater = 10 feet (3.0m)

B. Feasible Methods (Figure 5)

Primary	Secondary
1. Wells	1. Grout
2. Compressed Air	2. Freezing

C. Comments

1. Dewatering: - Required drawdown = 60^{+} feet (18.3m).

   - Fractured rock could supply a large volume of water

   - Stratified silt and sand may be difficult to predrain

   • Wellpoints - Not applicable in bored tunnel.

   • Ejectors - could be used to predrain to top of rock if volume to be pumped is not excessive. Would aid in stabilizing silt layers. Would not be cost effective to use in rock if tight spacing required.

   • Wells - If a large volume is to be pumped from the rock, wells would be the best tool. May also be effective in predraining silt and sand. If volume is small then open sumping would probably be sufficient.

2. Compressed Air/Slurry/EPB Shields

   - Possible problems excavating mixed face conditions under air pressure.

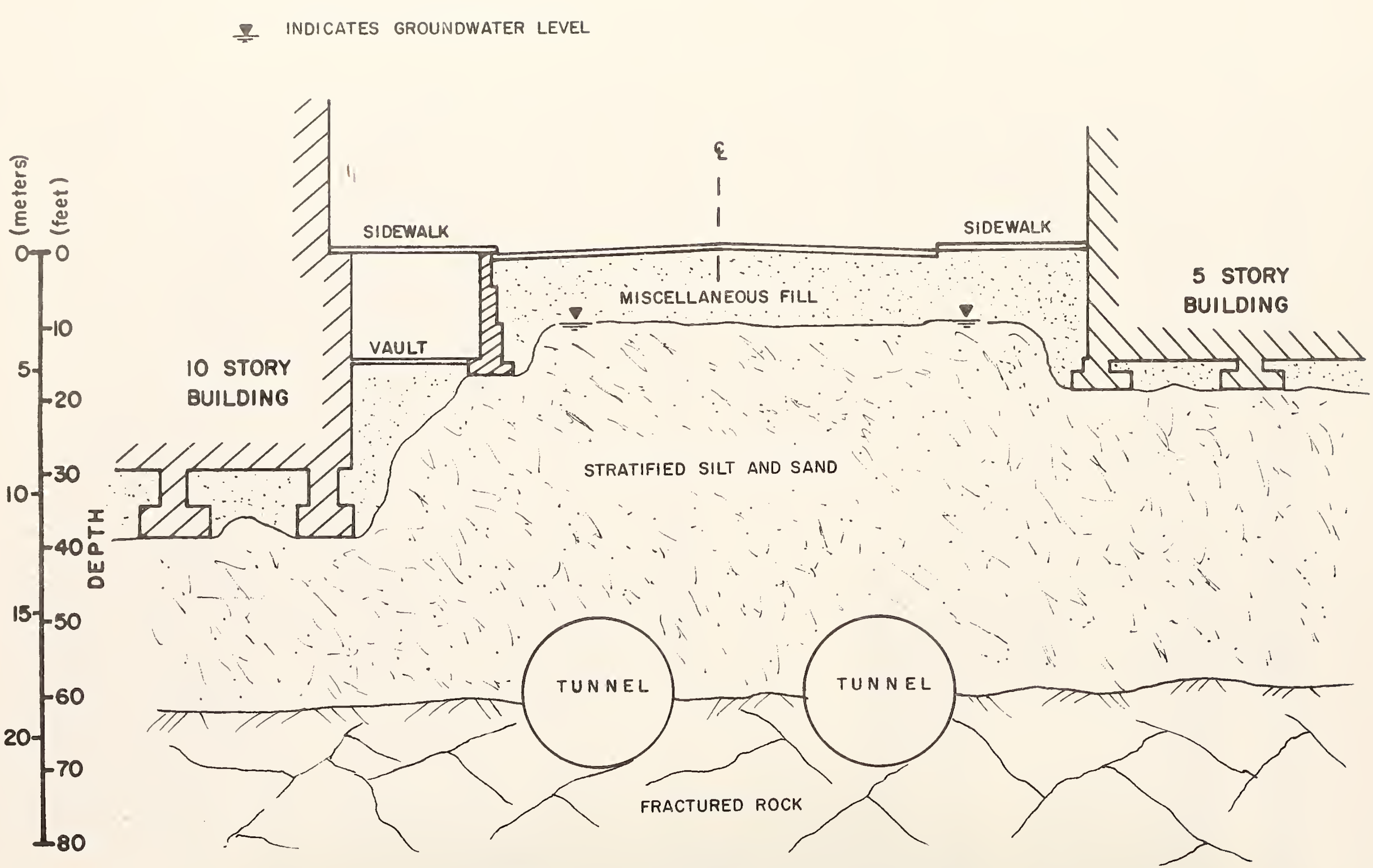


FIGURE 51 - Example 10, Cross Section

- Slurry or EPB shields not feasible because of rock in lower half of face.

3. Grouting

   - See Example 5

4. Cutoffs

   - See Example 5

5. Freezing

   - See Example 5

   - May not be able to freeze water in rock seams effectively if seepage velocities to high.

6. Summary Discussions

The use of a shield in mixed face conditions is difficult becuase of the different excavation techniques in soil and rock. This is particularly true if pressure relief is required for bottom stability. Deep wells can probably effectively drain the fractured rock zone. Depending on the permeability of the stratified sands and silts, wells or ejectors may be required to drain the stratum, with secondary pumping as necessary to handle residual seepage.

Grouting of the tunnel perimeter is possible if a particulate grout is used in the large rock seams and a low viscosity chemical grout is injected into the stratifed layers above the tunnel arch. Both freezing or cutoff methods are feasible but probably very expensive.

Compressed air with pressure reduction due to partial dewatering is also a feasible method. The compressibility and extent of the silt should be reviewed to determine recharge requirements.

4.90.11 EXAMPLE 11 - BORED TUNNEL IN FRACTURED ROCK

A. Project Conditions (Figure 52)

- 20 feet (6.1m) Diameter Bored Tunnel, in Urban Area
- Depth to Invert = 70 feet (21.3m)
- Soil and Rock: 10 feet (3.0m) Miscellaneous Fill

  10 feet (3.0m) Dense, gravelly sand

  100 feet (30.5m) Fractured Dolomite
- Depth to Groundwater = 10 feet (3.0m)

B. Feasible Methods (Figure 5)

Primary	Secondary
1. Wells	1. Grouting
2. Sumping	

C. Comments

1. Dewatering: - Required drawdown = 60$^+$ feet (18.3).

   - Depending on size, frequency and continuity of fracture pattern, volume of water could be large.

   • Wellpoints - Not applicable in bored tunnels

   • Ejectors - Not able to handle large pumping volumes

   • Wells - If large volumes are to be pumped, wells would be the best tool. If volume is small, sumping is suitable.

2. Grouting

   - Cement grout can be used in larger joints. Low viscosity chemicals grout can be used as a secondary treatment, but only in extreme cases.

3. Sumping

   - Possible if the rock is not badly fractured and flows are small.
   - Washing of joint filling material may cause instability.

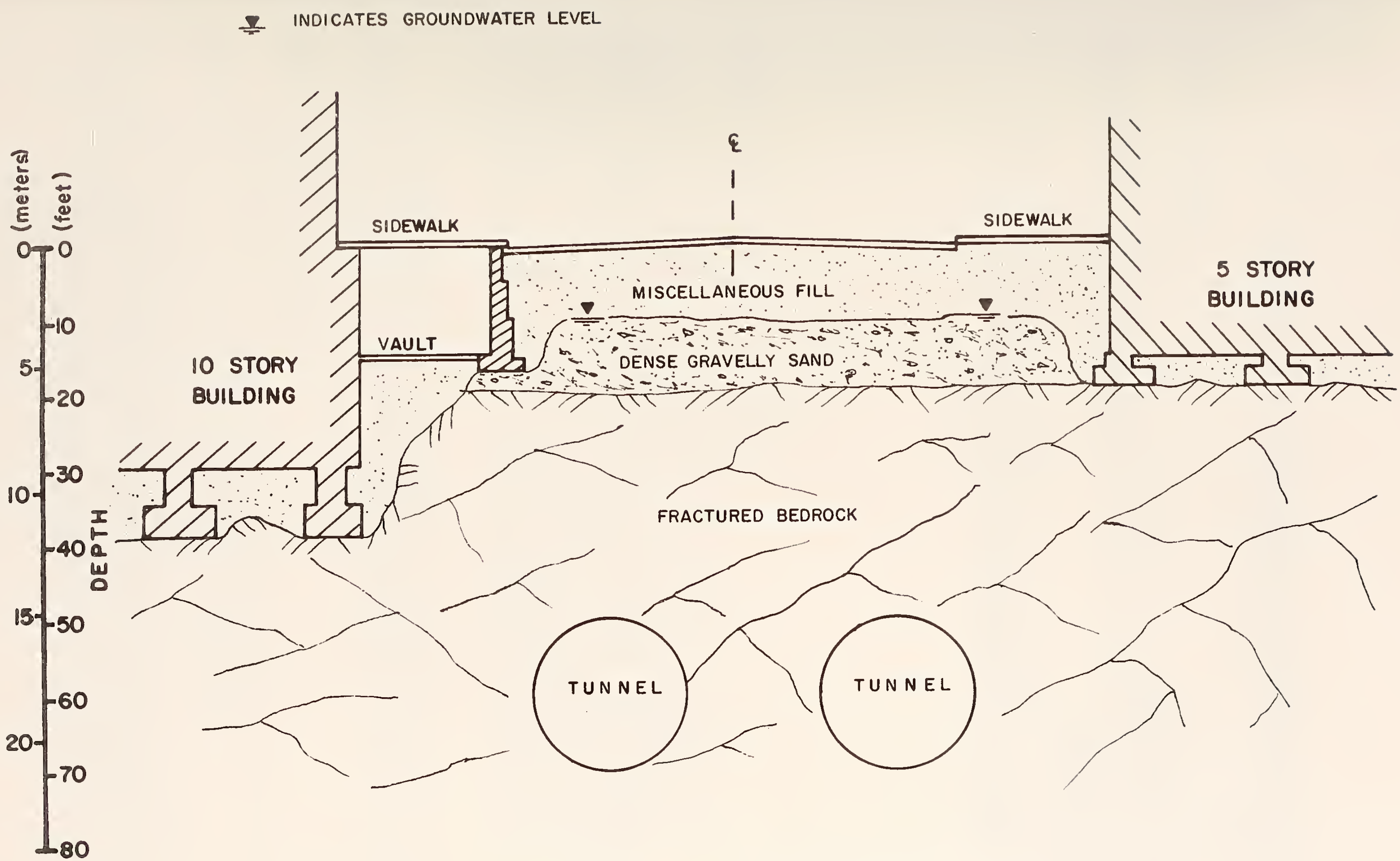


FIGURE 52 - Example 11, Cross Section

- Possible as a secondary drainage technique

4. Cutoffs/Freezing

   - Only justified in extreme circumstances

6. Summary Discussions

Wells, sumping or grouting are all feasible methods. Which will work best in a function of expected flow rate, degree of rock deterioration and fracture pattern.

5.00 DESIGN AND CONSTRUCTION OF SYSTEMS TO PREVENT INTRUSION OF GROUNDWATER INTO COMPLETED TUNNELS

5.10 GENERAL

This section summarizes methods of design and construction of systems to prevent groundwater intrusion into completed tunnels included is discussion of:

- Cast-in-place Concrete
- Applied Waterproofing Envelopes
- Segmented Linings
- Grouting
- Cut-off Walls
- Sunken Tube Tunnels

5.20 CAST-IN-PLACE CONCRETE

The creation of impervious concrete requires rigid control of all materials and processes which can add greatly to the cost and is usually difficult to achieve in adverse tunnel environments. Water will readily pass through joints, cracks, honeycombs, and the pores of the intact concrete. To produce impervious concrete, all joints must be carefully and permanently made watertight. Designs must be such that cracks do not develop; honeycombing must be prevented by careful concrete placement; and all pores capable of transmitting water must be filled or isolated.

If settlement is anticipated, elastic membranes can be used to effectively bridge cracks. Use of at least twice the minimum longitudinal reinforcing steel required by ACI for shrinkage is recommended to minimize cracking.

5.21 Concrete Design

The design of watertight concrete should follow the best practices for high quality structural concrete.

The aggregate should be clean, sound, not reactive with the cement, free from organic and other deleterious materials and sized appropriately for the section to be placed. It should be well graded to minimize voids. Portland cement should be finely ground and of the type best suited to project conditions.

Water should be free from deleterious amounts of akalies, acids, chlorides, oils, and organic materials. The water-cement ratio should be as low as possible consistent with good workability to reduce permeability and to increase shrinkage resistance. Admixtures should be compatible with the other constituents of the mix. Careful attention to shrinkage resistance is especially important if waterproof concrete is desired. Shrinkage decreases with decreasing water-cement ratio, which can be reduced by the addition of chemical admixtures such as Plastocrete by Sika or, the addition of finely divided materials. Shrinkage can be controlled by the use of an expansive cement or admixtures such as finely divided aluminum filings. Thermal shrinkage can be reduced by chilling constituent materials of the plastic concrete before placing; by replacing a portion of the Portland cement with a material such as pozzolan which will act as a cement but does not produce as much heat during its hydration or by spraying cooling water on the forms after concrete placement and curing with cool water sprays.

Improvement of chemical resistance requires determination of the environment, both external and internal, to which the concrete will be exposed. The mix can be made resistant to many aggressive chemicals by altering constituents such as the use of Type II or V cement for sulfate resistance.

Concrete must be compatible with other materials embedded in it such as aluminum, which reacts with the alkali hydroxides in the cement forming a compound which occupies more space than its constituents and therefore, cracks or spalls the concrete. Iron oxides are also expansive in concrete. Aluminum items must be coated, but iron need not be unless oxygen will be available to it.

Adequate impervious cover of reinforcing steel is required to prevent corrosion. The relationship between the thickness of cover over reinforcing steel and permeability of concrete is illustrated in Figure 53.

To minimize the effects of stress corrosion, and to reduce shrinkage cracks intermediate grade steel should be used with higher than the theoretical design area. Cathodic protection may be necessary if ground conditions are particularly corrosive.

5.22 Concrete Construction

Some key practical considerations include:

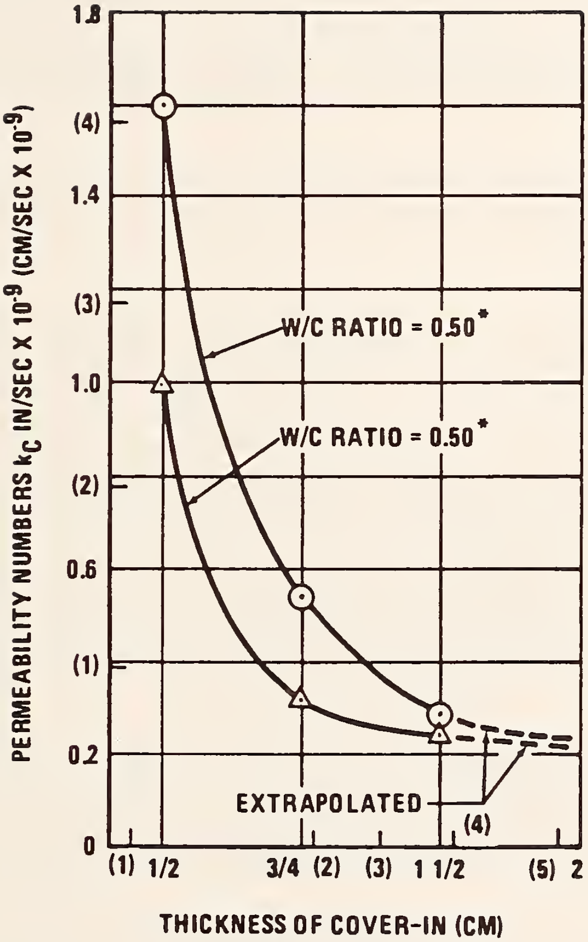


Figure 53- THICKNESS OF COVER AND PERMEABILITY
(From Birkmyer, Ref. 14)

1. Thorough mixing improves plasticity, resulting in better placement and uniform distribution of the constituent materials. The fine and coarse aggregates must be mixed to produce as compact a mass as possible with the cement particles.

2. If there is a delay in placing the concrete, aggregates may settle out, causing segregation and producing permeable concrete. Placement delays will also reduce the plasticity of the mix. Addition of water to restore the plasticity will produce a more permeable concrete.

3. Pumping concrete long distances tends to dry out the mix and thus an admixture may be required to aid the flow.

4. If concrete is placed against the ground surface, there should be no debris or projections into the design volume.

5. There should be no standing water present prior to concrete placement. Running water should be controlled so that it will not mix with the concrete.

6. If pouring against hardened concrete, the hardened concrete shuld be well roughened, cleaned, moistened, and treated with cement mortar just before placement.

7. Deposit concrete as nearly as practical in its final position and place it in shallow lifts so that they can be well compacted. It should be possible to reach the top portion of the plastic lift below by the internal vibrators.

8. Vibrate the concrete to compact it thoroughly using internal vibrators or vibrators attached to the forms or both. Use special care in compaction along joints and around the reinforcing steel, and other embedded items.

9. Once pouring has begun on a section, it should be completed without interruption.

10. Moist curing is essential to obtain watertight concrete. If the concrete dries before sufficient hydration has taken place, drying shrinkage will occur. A minimum of seven days of water or fog curing should be required. After moist curing, the concrete should be protected from excessive drying conditions such as those caused by dry air draughts.

11. To ensure watertightness of the finished structure, all defects such as honeycombing, bolt holes, pop-outs, and cracks must be repaired. Repairs to damaged or defective concrete should be performed within 24 hours since the bond with green concrete is generally stronger than that with fully hardened concrete. Tie holes and pop-outs must be filled and sealed; all porous concrete (due to honeycombing, aggregate segregation, etc.) must be removed and replaced with sound concrete, and all cracks must be sealed. The American Concrete Institute and the Bureau of Reclamation have published procedures for the repair of surface defects which are quite comprehensive. Several approaches to repair of defects are illustrated in Figures 54 through 56.

12. Various resins such as epoxy and polyester, as well as synthetic rubber and bituminous materials have proved helpful in bonding and sealing defective concrete.

5.23 Construction Joints

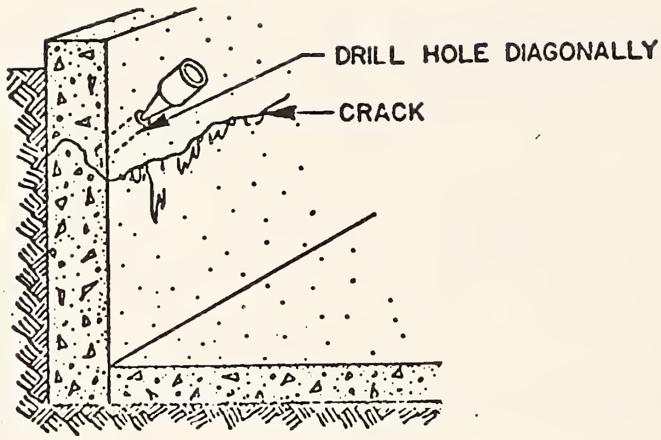
Construction joints are required due to placing limitations, such as between slabs and walls and are intended to function as though no joint were there.

Reinforced construction joints require that the fresh concrete bond perfectly with the hardened concrete so that the joint will be watertight. Extra precautions are usually required such as the installation of a waterstop across the joint or bentonite backing outside the joint. With slight modification a bentonite sealed joint can be used when concrete is placed against the excavation or excavation support. Bentonite applications are simpler to apply than waterstops and have the advantage of being self-healing. Typical construction joint details are shown in Figures 57 and 58.

5.24 Contraction Joints

Contraction joints are designed to open when the concrete shrinks. They control the location of shrinkage cracks which may have to be bridged with an elastic membrane, or waterstop to prevent leakage.

Reinforcing steel crossing a contraction joint is treated with a bond breaker so that as contraction occurs there is less tensile strength at the joint making it a favorable location for a crack to occur. The weakness of this plane may be further enhanced by chases along each face of the concrete as shown in Figure 59, which are then waterproofed.



Repair of crack

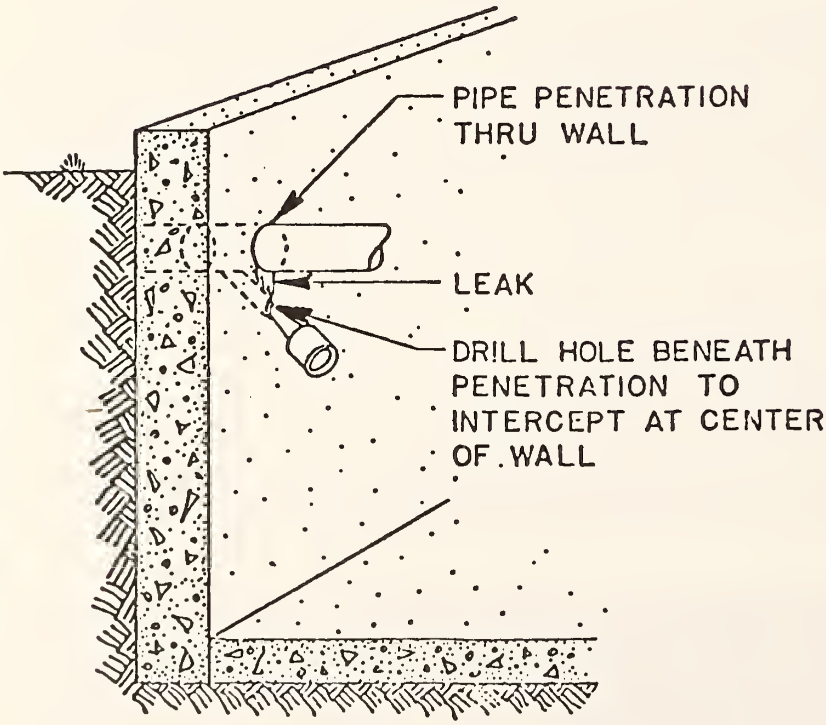
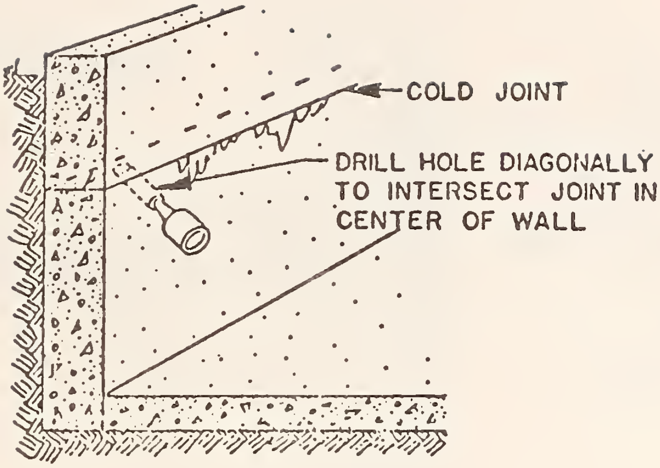


Figure 54 - Repair of Wall Defects - Cracks
(Ref. 69) (From Birkmeyer, Ref. 14)



Repair of cold joint

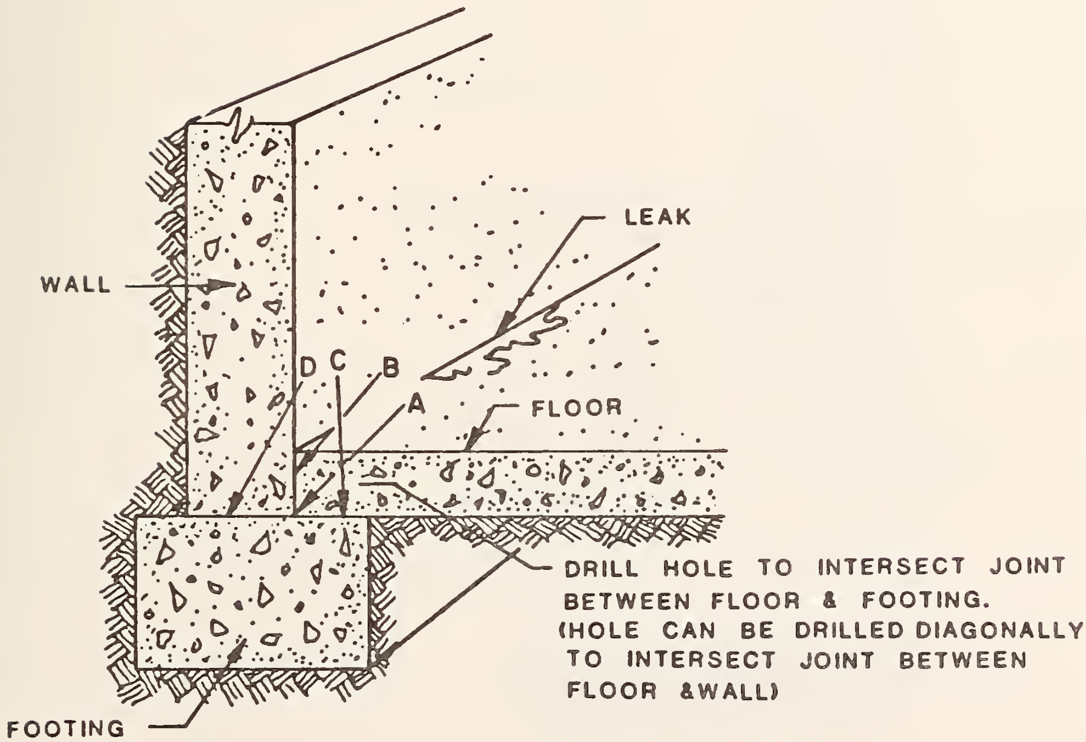
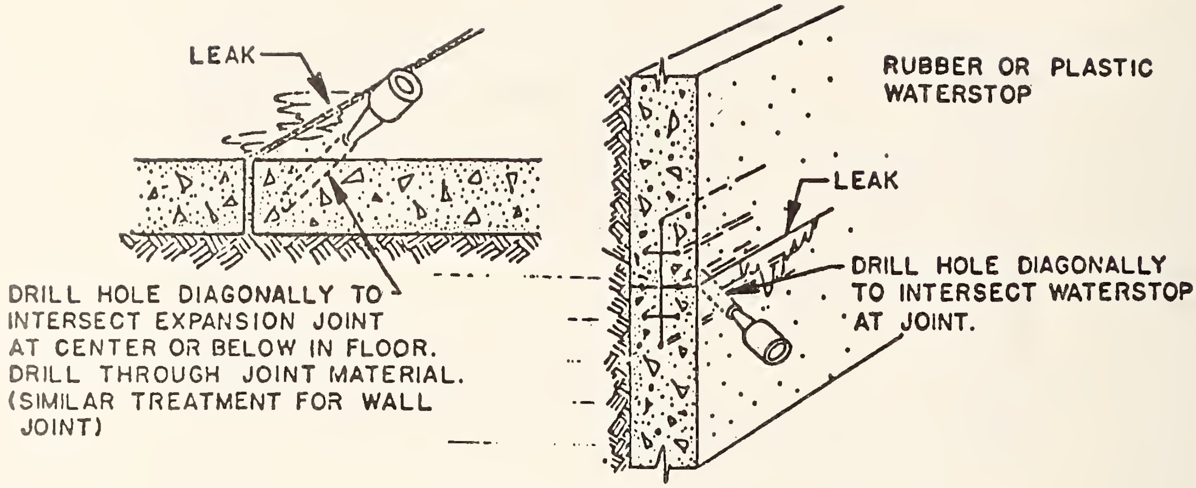


Figure 55 - Repair of Wall Defects - Cold Joints (Ref.69)



Repair of expansion joint and waterstop in concrete structure

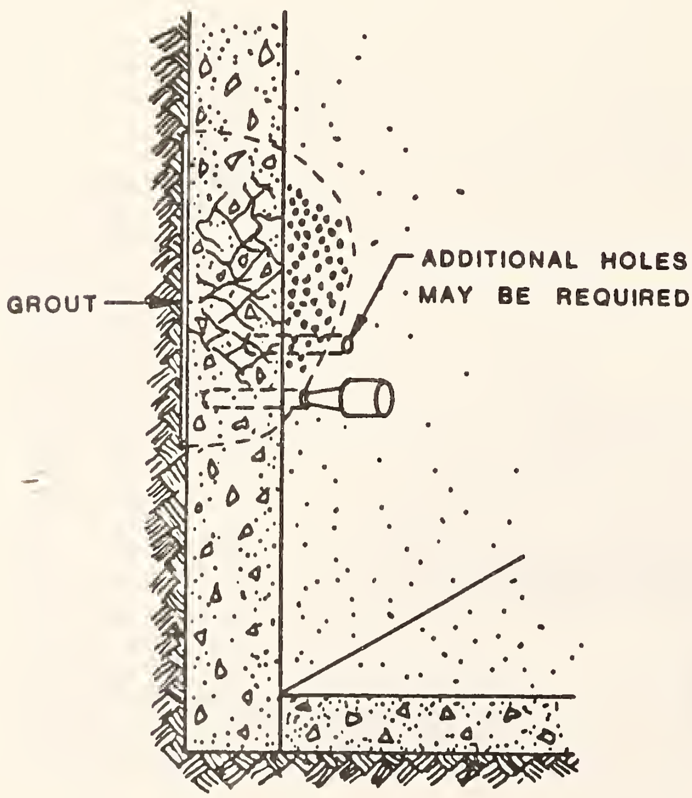
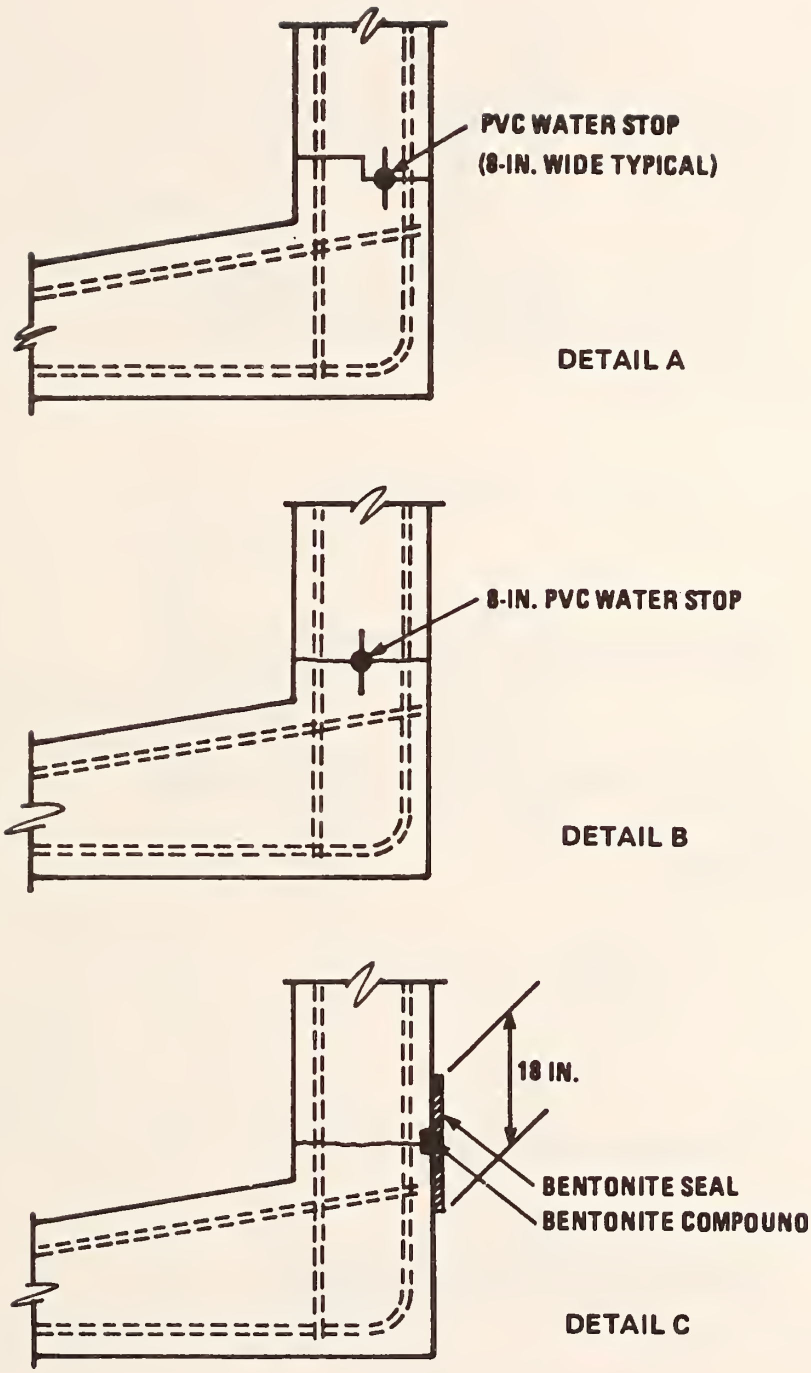
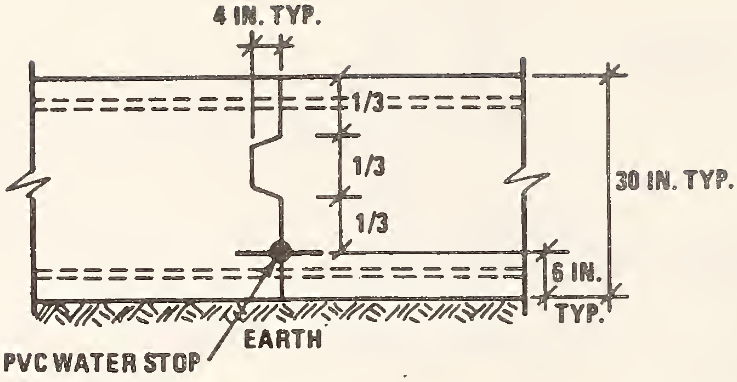


Figure 56 - Repair of Wall Defects - Expansion Joints
(Ref. 69)

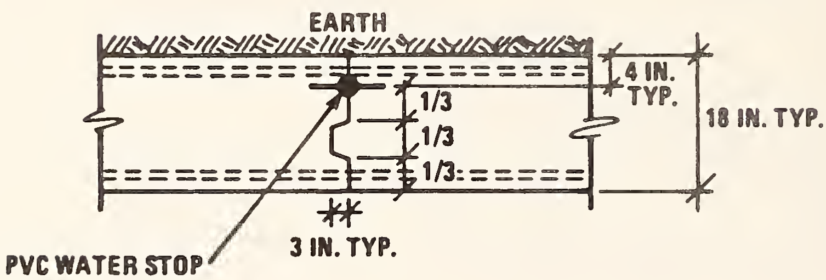


(1" = 25.4 mm.)

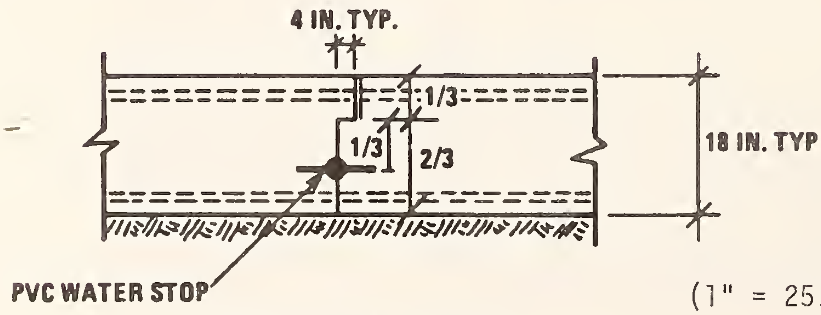
Figure 57 - Construction Details - Longitudinal Joints



DETAIL A – INVERT JOINT



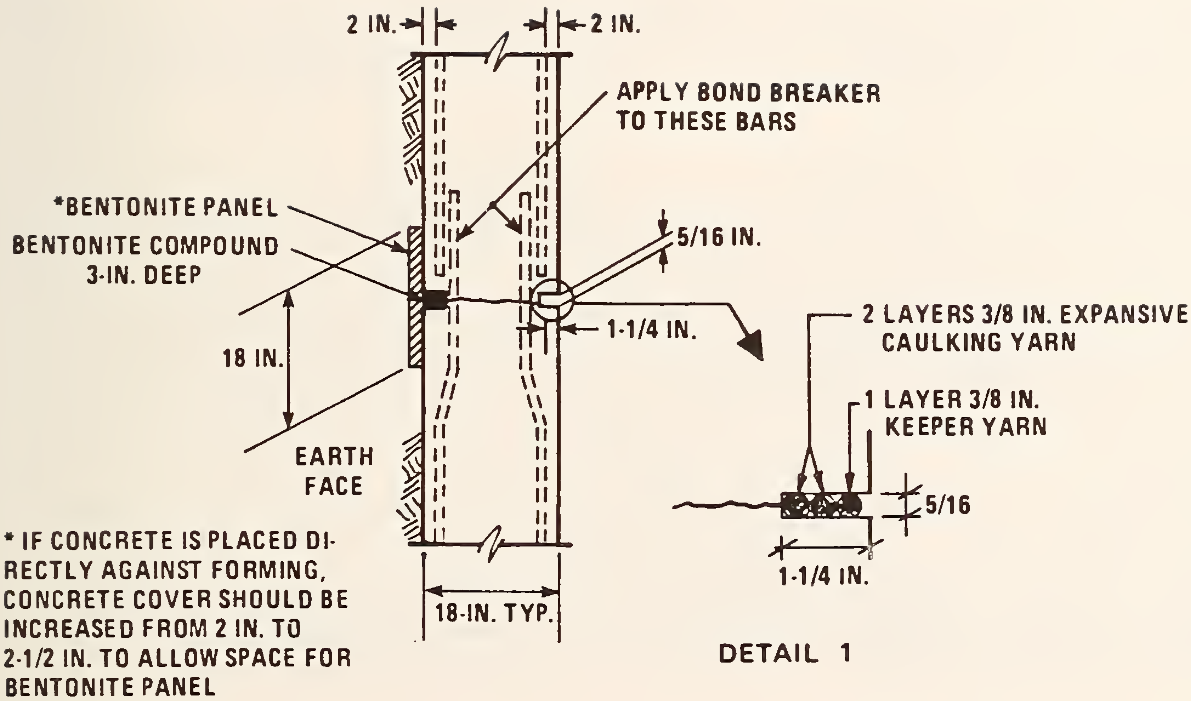
DETAIL B – ROOF JOINT



(1" = 25.4 mm.)

DETAIL C WALL JOINT

Figure 58 - Construction Details - Transverse Joints



(1" = 25.4 mm.)

Figure 59 - CONTRACTION JOINT DETAIL
(From Birkmyer, Ref. 14)

5.25 Expansion Joints

Expansion joints are seldom necessary in tunnels except near entrances where temperature variations may be substantial. They prevent excessive compressive stress in the concrete by allowing unrestricted movement as temperatures increase.

No reinforcing steel crosses expansion joints. Usually a premolded compressible joint filler is placed against the hardened concrete and fresh concrete is placed against the joint filler. When completed, a joint sealer may be applied at the surfaces. As the concrete expands the joint filler is compressed reducing the compression stresses that would otherwise develop in the concrete.

If an expansion joint is required below the water table, a waterstop should be used to prevent leakage. It is very important to be sure the center bulb is not bonded to the concrete and is centered in the joint.

5.26 Waterstops

Waterstops are metal (rarely used), rubber, or plastic strips placed in joints to prevent the flow of water when the joint opens. Waterstop materials must be able to withstand extensive elongation and return essentially to the original configuration. Various cross sections have been developed so that the strip holds in the concrete when the joint opens. Waterstops are typically between 3 and 12 inches (75 and 300mm) wide and 3/32 to 1/22 inch (2 to 13mm) thick. The dumbbell configurations are customarily used for rubber waterstops. If considerable differential movement is anticipated, a waterstop having a thin, weak, longitudinal section can be used. This will split when movement occurs, thereby allowing a differential movement of approximately the bulb circumference without rupture. Typical configurations of metal, rubber, and PVC waterstops are shown in Figure 60.

5.30 APPLIED ENVELOPES

Waterproofing envelopes may consist of any of a number of materials or combinations of materials. The most common are:

1. Built-up membrane consisting of hot-applied bituminous cementing materials alternating with fabrics or felts saturated with similar bituminous material.

2. Preformed multi-layered boards.

3. Plastic and synthetic rubber sheets.

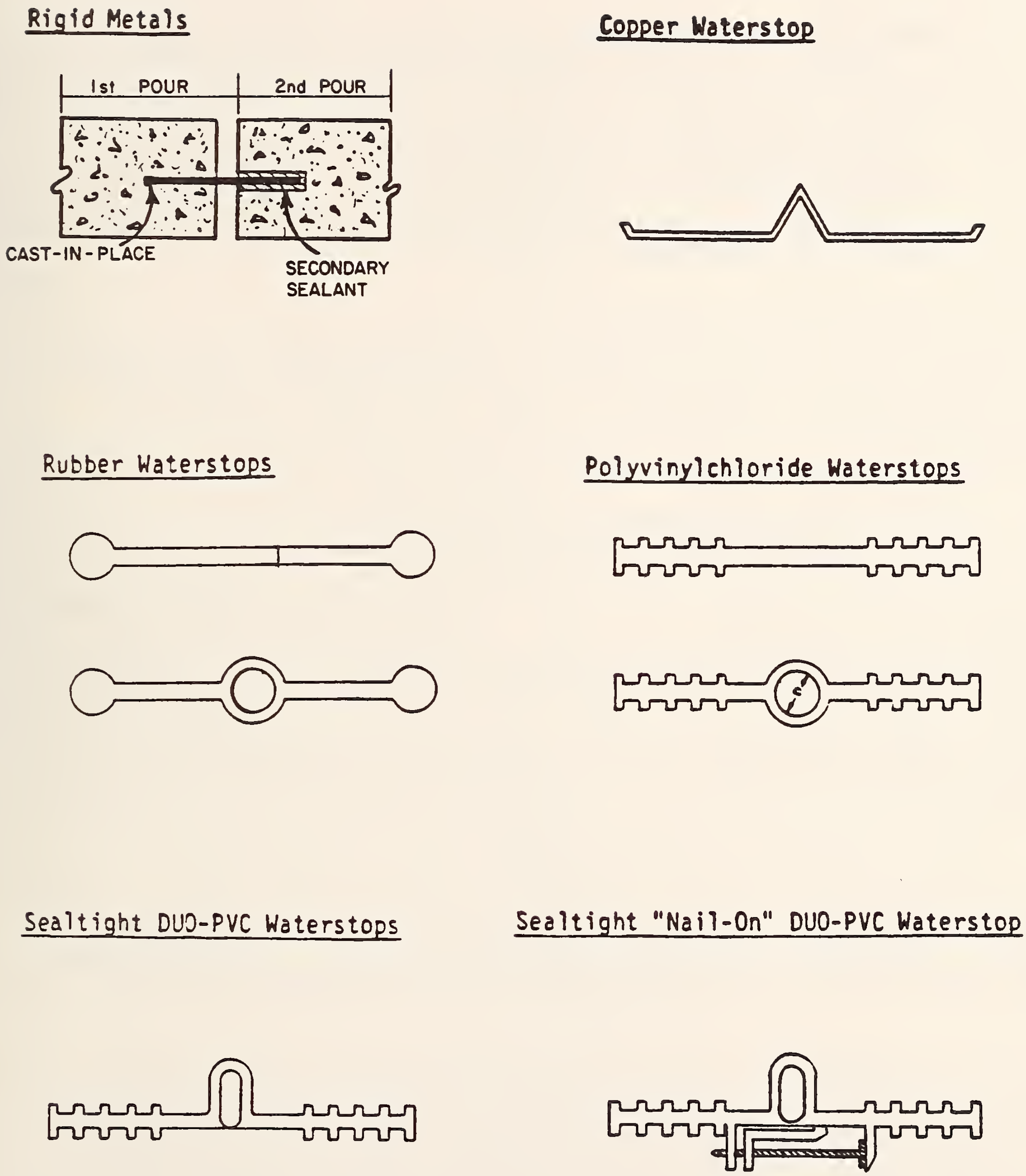


Figure 60 - TYPICAL WATERSTOP CONFIGURATIONS
(From Stilling - Ref. 119)

4. Bentonite clay applied in sheets or sprayed on.

5. Cold, liquid-applied membranes, especially the elastomers.

6. Cementitious coatings applied to the inside of the structure by the plaster-coat method or the iron-coat method.

7. Brick set in asphalt mastic.

8. Corrugated aluminum sheeting.

Membrane waterproofing systems must accommodate any cracks in the concrete. If earth is to be placed against a surface to be waterproofed, the membrane must also resist damage due to backfilling, either by itself or by the addition of a protective covering.

5.31 Built-up Membrane Waterproofing Systems

Built-up membrane waterproofing consists of a series of alternating applications of a cementing material and plies (sheets of woven fabric or felt, or a mat). As the plying cement for hot-applied bituminous-based built-up waterproofing the choice is generally between properly prepared natural asphalt, asphalt derived from asphaltic petroleum, coal tar pitch, and coal tar enamel. Coal tar pitch and coal tar enamel (with mineral fibers added to make the material more stable) have performed well buried in soils for 50 to 100 years and are preferred to asphalt products.

The plies may be of woven organic or inorganic fibers, felted organic or inorganic fibers, or mats of uniformly distributed glass fibers. These sheets are then impregnated with bituminous material compatible with the plying cement used. Recently other types of plies have been developed for tunnel work such as the copper foils and jute burlap strips bituminized on both sides.

A 3-ply membrane system is frequently used. This system consists of a layer of hot bituminous material mopped on the surface to be waterproofed and followed immediately by the application of a "ply" of bituminous material; this is followed by a second layer of cementing material and another "ply", followed by another mopping with hot bituminous material completely covering the third "ply." The successive plies are so arranged that no two seams are coincidental and within each layer the sheeting is lapped a minimum of 1 inch (25mm).

The surface to which the membrane is attached must be clean, dry and reasonably flat. Figure 61 shows typical multi-ply systems specified by the American Railway Engineering Association.

Sagging of hot asphalt or coal tar on vertical surfaces can be partially overcome by performing the applications in sections and backfilling the section promptly to support the membrane. Another means of keeping it in place is to secure the plies to the wall so that they help to hold the hot tar in place.

Frequently, after the membrane is in place, a thin concrete layer is placed over the membrane to protect it from damage and then backfilling is accomplished.

To avoid stripping the membrane off due to soil settlement relative to the structure, the waterproofing membrane can be sprayed with a bituminous anti-friction layer having a thickness of 8 to 10 mm (0.3 to 0.4 in.) which allows the soil to slip easily along the wall without damaging the waterproofing.

5.32 Preformed Multi-Layered Board

Preformed multi-layered vapor and waterproofing materials are available as semi-rigid boards, usually 4 feet by 8 feet, (1.2m by 2.4m) and in rolls.

The material consists of a minimum of one sheet of reinforcing material embedded in an elastomeric material composed of bituminous and synthetic resin materials. An example of a multi-layered board is shown in Figure 62. The welding together of the various constituents is superior to that in the built-up membranes.

It is necessary to make sure the concrete is free of all foreign substances and loose surface material, any cracks have been repaired, any honeycombing has been chipped out and replaced by sound concrete, and all construction joints are tight.

The prepared concrete substrate is coated with the primer or a bonding agent. If the membrane is not the self-bonding type it is usually also coated with a bonding agent. Then the material is placed in position and rolled to make a good, complete bond with the substrate. The sealing of the joints in the membrane must be carefully done. Backfilling can usually be done immediately, though under certain conditions a protective course is recommended.

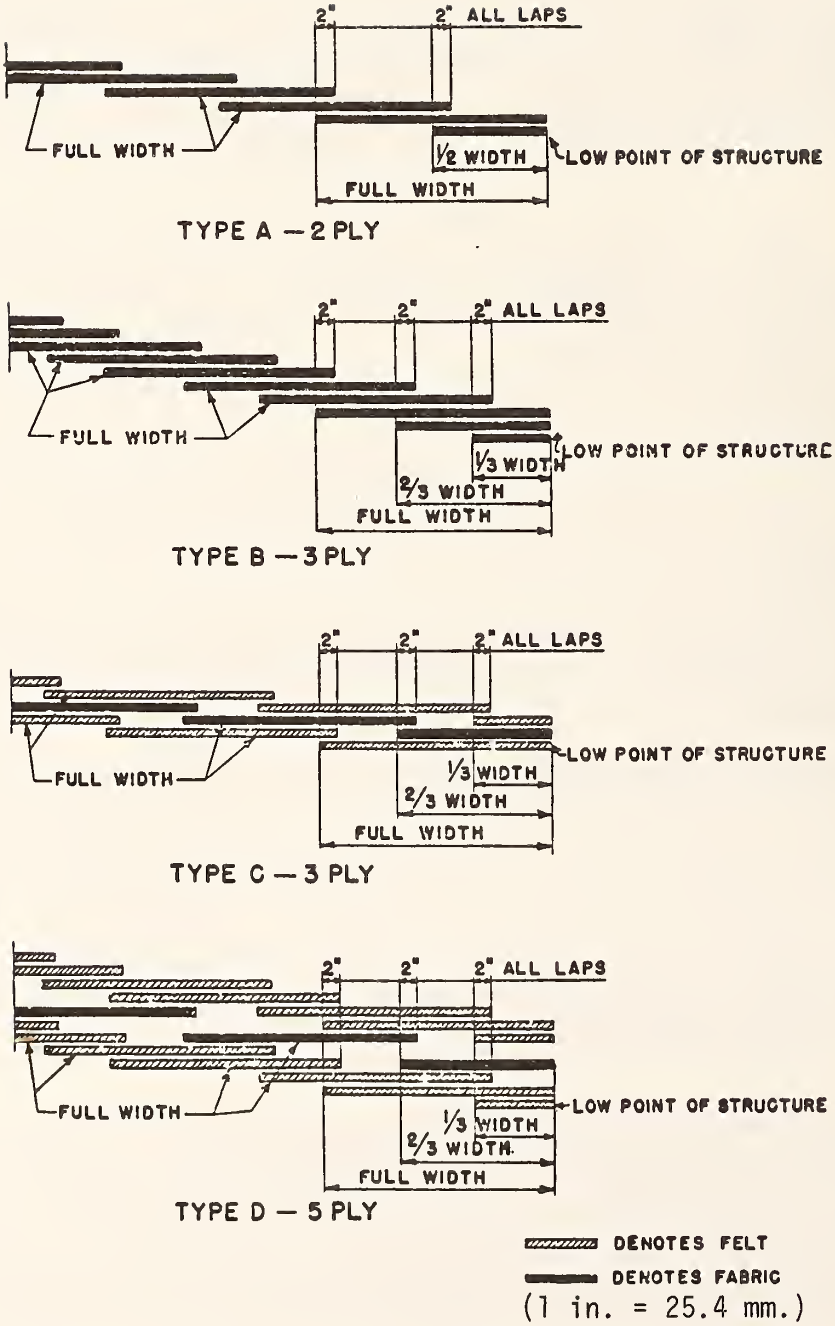


Figure 61- TYPICAL BUILT-UP WATERPROOFING MEMBRANE SYSTEMS (From American Railway Engineering Association Manual, Ref. 4)

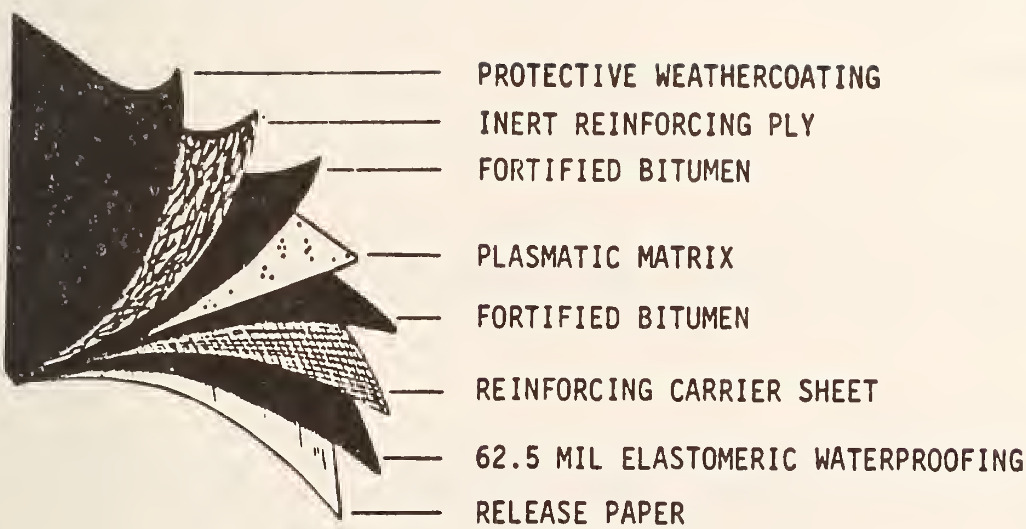


Figure 62 - TYPICAL PREFORMED MULTI-LAYERED BOARD SHOWING TYPES OF PLIES (From W. R. Meadows, Melnar product data sheet)

5.33 Plastic and Synthetic Rubber Sheeting

The materials which are most promising both in cost and properties are polyethylene, polyvinyl chloride, butyl rubber, Hypalon, and Neoprene. Their advantages over built-up membranes include:

1. Resistance to many chemicals
2. Resistance to bacteria
3. Superior elasticity and elongation before rupture
4. Resistance to aging
5. Wide temperature range of usefulness
6. Electrical resistance
7. Lightweight
8. Application possible without use of hot mastics
9. Resistance to puncture
10. Self-healing to some degree, especially the rubbers
11. Flexibility

Polyethylene Sheeting

Polyethylene sheets can be joined by heat fusion and by epoxy or other adhesives which makes it possible to attach the sheets to almost any surface.

The sheets may be applied to the outside of cut-and-cover structures, but must be protected. On walls this may be asbestos cement board, concrete plank, insulation board, concrete blocks, or common brick in mortar, or similar material. On inverts a 2-inch (50 mm) layer of concrete is usually sufficient and on the roof a minimum of 3 inches (75 mm) of concrete. If the roof will be subject to damage, other suitable protection should be employed.

If no space is available between the excavation support and the wall, the sheeting is placed against the form before the concrete is placed. All joints in the sheeting must be completely sealed and tested before backfilling or concrete placing is undertaken.

In tunneled structures the sheeting is usually placed between the primary lining, whether it be segmented lining, cast-in-place concrete, or shotcrete, and the secondary liner.

Polyvinyl Chloride Sheeting

Polyvinyl chloride (PVC) sheeting can be used similarly to polyethylene. Specially designed types of sheeting are available which can be fastened to the forms before placing the concrete. These have T-shaped projections which interlock with the plastic concrete and which are then integral with the concrete when it has hardened. An example of this type of material is B. F. Goodrich's Koroseal Lok-Rib.

PVC sheeting is expensive and is, therefore, infrequently used in tunnels except under extremely difficult conditions which require a mechanical bond with the concrete.

Adhesive and welding techniques are used for sealing the sheets of material to one another.

Butyl Rubber Sheeting

Of the synthetic materials, butyl rubber is the most impervious and frequently used product in the United States. It adapts well to surface irregularities due to to cold flow and will self-heal small holes such as those around puncturing nails or projections.

Butyl rubber is easily cut and fitted around openings, penetrations and other irregularities and can be spliced with cold self-vulcanizing butyl cement and/or unvulcanized butyl gum tape. Typical membrane splices recommended by the American Railway Engineering Association are shown in Figure 63.

The application of butyl rubber is accomplished in much the same way as plastic waterproofing sheeting, but the rubber is highly abrasion resistant and rarely needs protection during backfilling or concrete placement.

Hypalon Sheeting

Hypalon is a chlorosulfonated polyethylene manufactured by DuPont which can be vulcanized at room temperature into a tough, chemically resistant rubber. Hypalon has the greatest elongation at break of the synthetic materials

(1" = 25.4 mm)

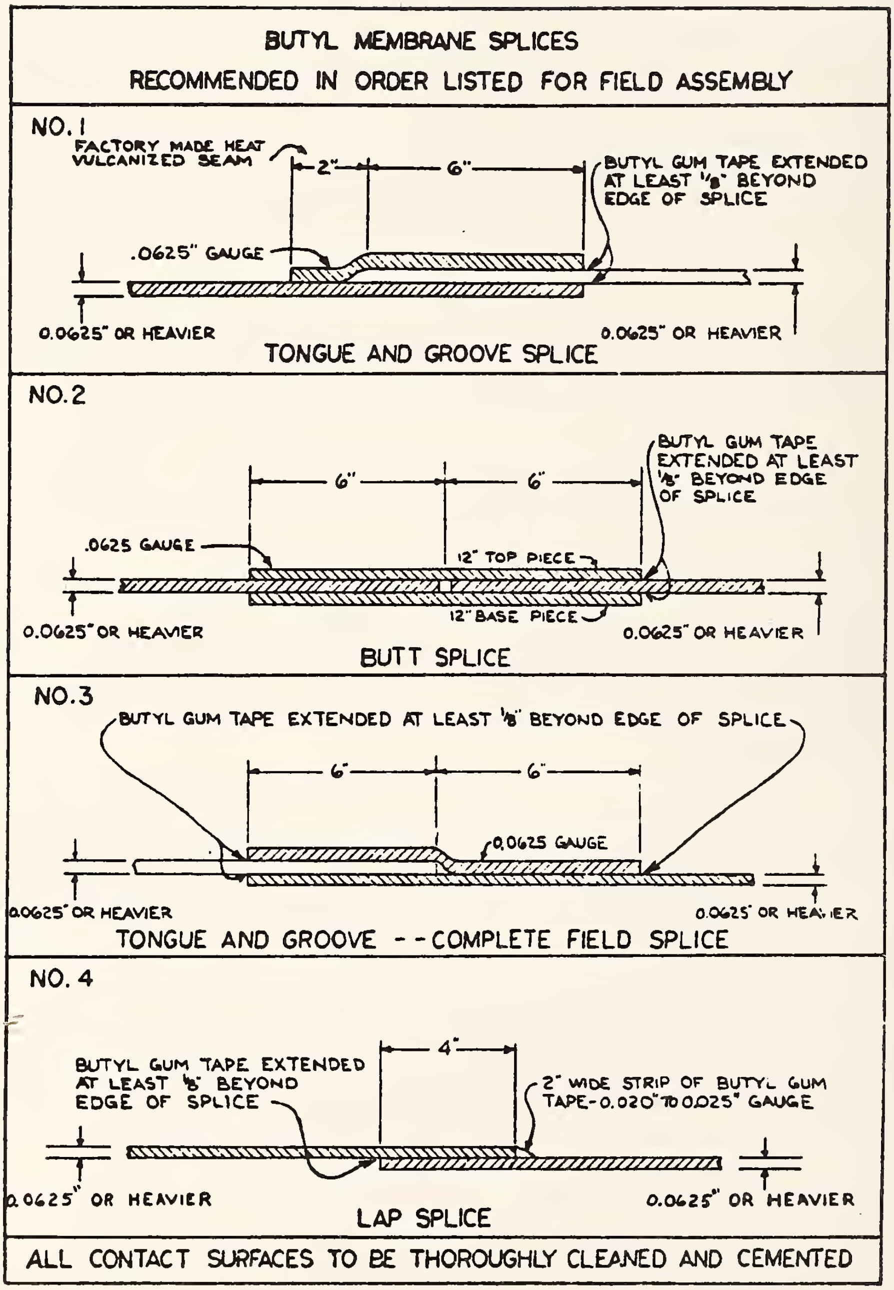


Figure 63 - TYPICAL MEMBRANE SPLICING DETAILS (From American Railway Engineering Association Manual, Ref. 4)

commonly used in waterproofing and, therefore, can successfully accommodate more differential structural movement than the other materials. It is, however, one of the most expensive sheetings currently available.

Neoprene Sheeting

Neoprene, the DuPont trademark for a chloroprene rubber, has properties similar to natural rubber, but can resist burning and biodegradation. It can be vulcanized in-place, or the sheets may be prevulcanized, to obtain the required properties.

5.34 Cold, Liquid-Applied Waterproofing

If leakage occurs through slightly porous concrete and the head of water is low, a coating on the inside of the structure may be sufficient to stop the leak. This is not good practice for complete watertightness, but is rather a remedial measure.

Used as a membrane on the outside of a concrete structure or sandwiched between two lining layers, many of these materials can do an effective job. Sprayed or rolled onto the surface they are less expensive and faster to apply than sheet materials and can much more readily be applied to irregular surfaces.

A disadvantage of cold, liquid-applied waterproofing is the difficulty in obtaining a uniform thickness throughout the coating and preventing pinholes. Several separate applications may be required which defeats some of the advantage of rapid placement.

5.35 Bentonite Panels and Spray

Bentonite is an excellent waterproofing barrier, applied either in panels or sprayed on the surface. There are two manufactures of bentonite waterproofing materials in the United States; American Colloidal Company, manufacturers of bentonite panels; and Effective Building Products, Inc., manufacturers of a spray applied bentonite material.

Bentonite Panels

Bentonite panels consist of granular bentonite sealed inside a smooth face sheet of biodegradable corrugated kraft paper. These panels are usually 4 feet (1.2 m.) square and 3/16 inch (0.5 cm) thick. The panels may be installed at any temperature on the finished concrete, on lagging, or on any other reasonably smooth surface against which the concrete will be placed. Typical bentonite panel installations are shown in Figure 64.

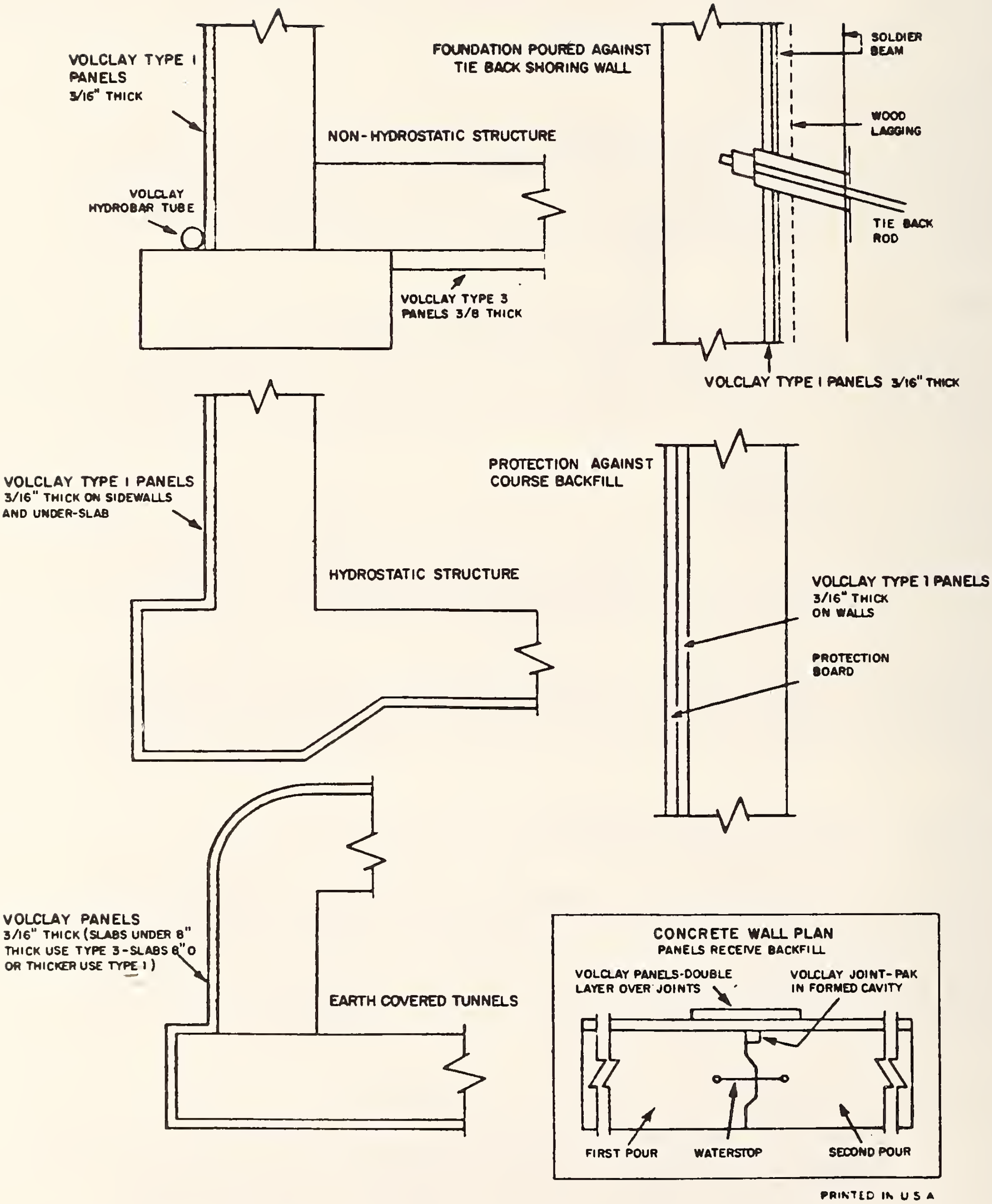


Figure 64 - TYPICAL BENTONITE PANEL INSTALLATIONS (From American Colloid Company's Volclay Panels brochure)

If the invert slab is to be waterproofed using these panels, special precautions such as a lean concrete working mat will be necessary to prevent tears. If the invert is wet or very damp, one or more layers of 4-mil (0.1-mm.) polyvinyl sheeting should be placed between the ground and the panels to prevent premature swelling of the bentonite.

The panels should be stapled together or secured in some manner so that subsequent placing of concrete will not dislocate them. If concrete cannot be placed immediately, the panels must be protected from moisture, foot traffic, and construction operations.

Backfilling must be done with care to protect the kraft paper from damage and movement. If the backfill contains coarse or irregular gravel, 1/4-inch (0.6 cm) hardboard or similar material may be needed to preserve the integrity of the panels during the backfilling operation.

When applied to the roof of a structure the panels must be covered with a minimum of two feet of compacted backfill or an equivalent layer of concrete or other material to prevent movement of the bentonite by heavy traffic. The panels should be secured to the roof so that there will be no movement during the backfilling operation.

It is usual to place a 6-inch (15 cm) layer of sand as a drain channel over the panels before placing and compacting the backfill.

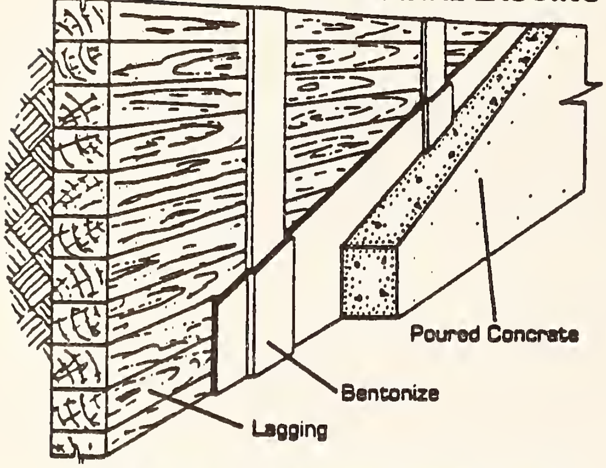
Bentonite Spray

An advantage of the spray-applied product is that the bentonite may be applied to irregular surfaces such as block masonry and spalled, honeycombed, or oozed concrete surfaces.

Bentonite spray is typically applied in a 1/4 to 3/8 inch (0.6 to 0.9cm) layer. Typical applications of the sprayed-on bentonite and associated products are illustrated in Figure 65.

Care should be taken in placing concrete and backfill against the waterproofing to prevent these materials from striking the surface and scouring the waterproofing off. Ideally, backfill or placing of concrete should follow application immediately. If this is not possible, the spray applied product requires the same sort of protection as do the panels.

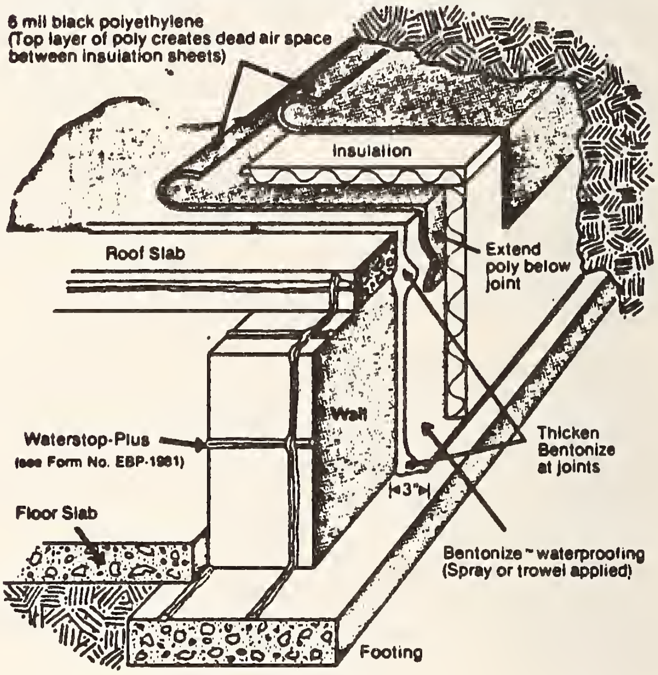
WOOD OR METAL LAGGING



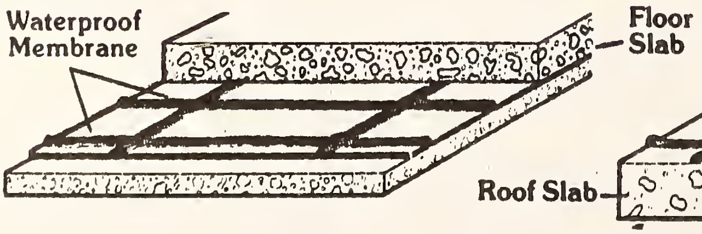
The Bentonize™ Waterproofing System used on a lagging type operation.

COMPLETE VERTICAL AND HORIZONTAL WATERPROOFING

The Bentonize™ Waterproofing System may be used on horizontal, vertical, or tunnel applications, including projects that require complete envelope sealing.

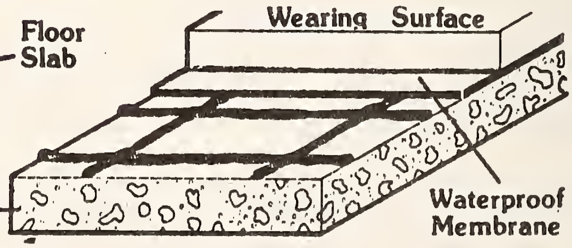


FLOOR SLAB



Leak Localizer is placed over the waterproof membrane in a square grid pattern to isolate any water leak coming through the waterproof membrane.

TUNNEL PLAZA OR UNDERGROUND ROOF SLAB



Leak Localizer placed in a manner which will isolate and stop the horizontal migration of water under the waterproof membrane, facilitating easy repair.

Figure 65 - TYPICAL INSTALLATIONS OF BENTONITE SPRAY AND ASSOCIATED PRODUCTS
(From Effective Building Products, Inc. Data Sheets)

5.36 Cementitious Waterproof Coatings

Plaster coat waterproofing and a variation of it, iron coat method, can be applied on the inside of concrete structures. They can be applied on wet surfaces and localized leaks can be repaired by a reapplication of the waterproofing coating. The concrete surface to which the coating is to be applied must be clean and well roughened so that there is good adhesion. The coatings must be moist cured and require skilled workers for proper application.

Plaster Coat Method

The plaster coat usually consists of a mixture of cement, sand, and possibly a waterproofing admixture which is troweled on the prepared surface to a depth of 3/4 to 1 inch (20 to 25 mm). If leaks occur, the coating may be removed and replaced or greater thicknesses used.

Iron Coat Method

Solvin's iron coat method of waterproofing consists of a mixture of portland cement, fine aggregate, water, pulverized iron, and a chemical oxidizing agent. Recesses, cracks, and intersections of vertical and horizontal surfaces should be packed with a grout containing the pulverized iron.

Three brush or sprayed coatings are applied and must be moist cured or the iron coat can be troweled on. Usually a protective coating is applied consisting of cement, water, and sand, to cover the "rusty" coat. In some applications, alternate brushings of the pulverized iron with the oxidizing agent and the iron-cement slurry are used.

If done well, the iron coat method can withstand a water head of 70 feet (21.3 m.).

Non-Metallic, Nonshrink, Cementitious Coating

The United States Grout Corporation has developed a non-metallic, nonshrink, cementitious coating which can be troweled, brushed, or sprayed on interior surfaces. It is a fast-setting, hydraulic cement-based mortar and is said to be superior to the iron coat method. It does not require skilled labor and a single 1/8 inch (0.3 cm.) thick coat (0.3 cm.) is recommended. The manufacturer's published data states that this 1/8-inch thickness will resist a 50-foot (15.2m) head of water.

5.40 SEGMENTED LININGS

Segmented linings of many types, shapes, and materials have been used, but the construction procedure is similar. A continuous complete lining is erected in the tail of a shield, or TBM one ring at a time. Historically, rings have been fabricated from one of three materials; cast-iron, steel, or concrete. Figure 66 depicts typical tunnel channel segments of each of these materials.

Cast iron is used infrequently, if at all, in the United States today. A detailed description of cast-iron segments is presented in Volume 2.

Fabricated steel segments are generally manufactured to close tolerances and are therefore reasonably water tight when bolted together. Caulking grooves and grommeted bolt holes are usually provided to improve water tightness as illustrated in Figure 67.

In a situation where a secondary cast-in-place concrete lining is required, the sealing of segments can be less stringent.

In a compressed air driven tunnel where the secondary lining is to be placed behind the face while the tunnel is still pressurized, no special precautions are needed (unless there is considerable air leakage).

There is a greater variety of configurations and sealing methods available for precast concrete segments than for cast iron or steel. Concrete segments are susceptible to leakage through cracks caused by improper handling, shield shove loads, and ground loads.

Although the concrete in precast segments is generally superior to cast-in-place concrete and the art of casting with steel forms and controlled materials results in very accurately dimensioned segments, they cannot match that achieved by machined metal plates. To compensate for this, designers of concrete segments often specify a sealing strip or gasket on all flange faces in addition to caulking. Other designers have come up with composite segments, basically of concrete, but with embedded steel plate or angles for flange or bolt bearing surfaces.

Various types of adhesive sealers have proven effective for low heads of water, whereas higher pressures require caulking and/or gasket seals on joints that are rigidly bolted (as with steel and cast iron). Asbestos-cement or epoxy-elastomerics are most commonly used.

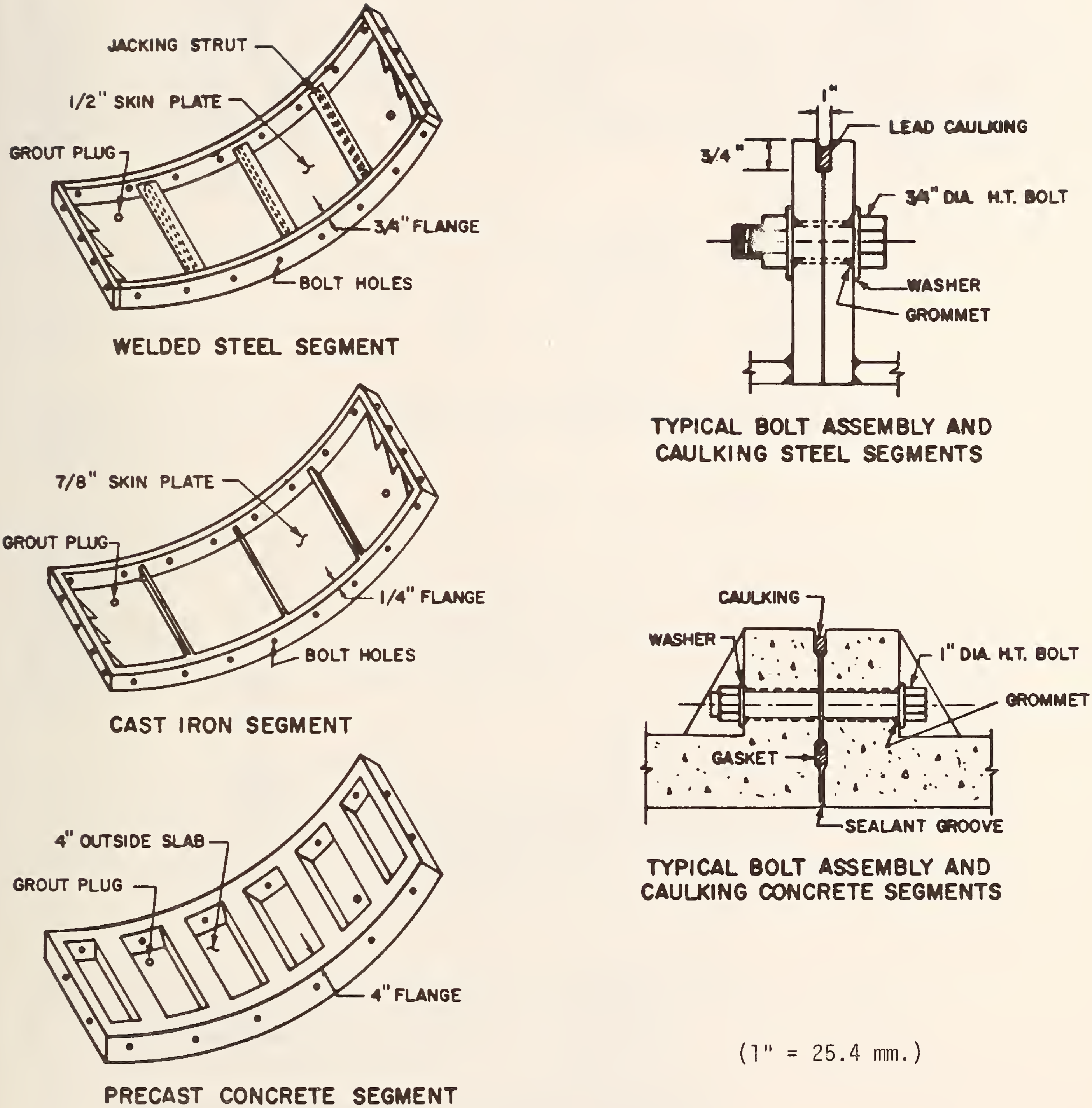
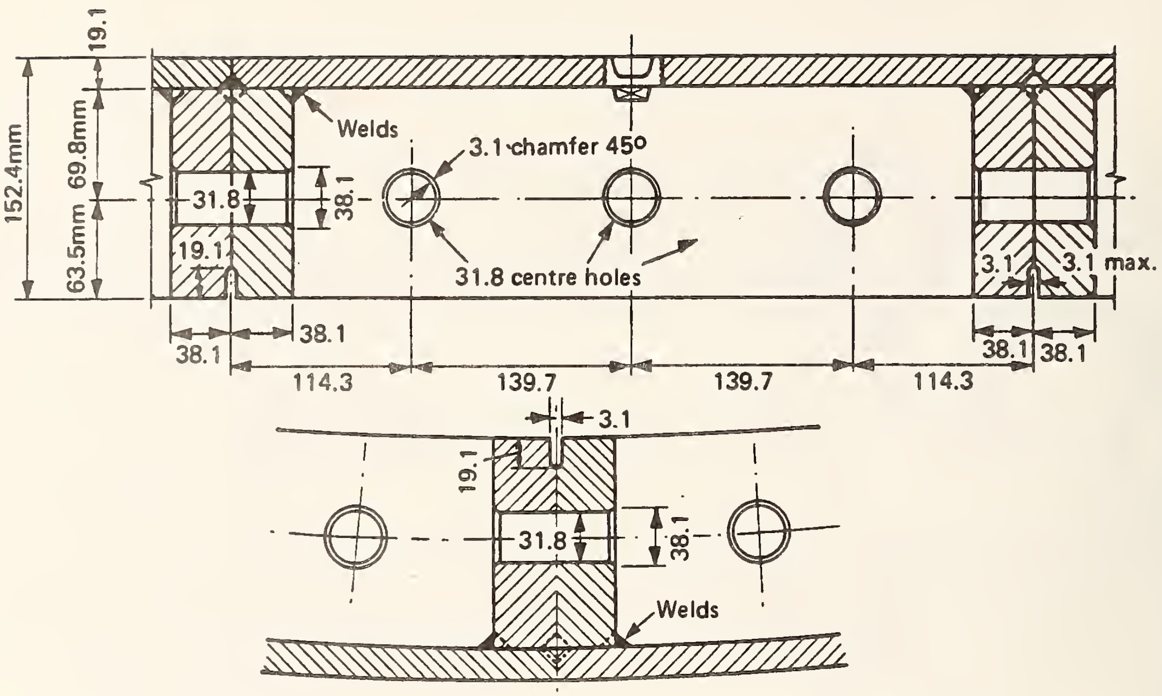
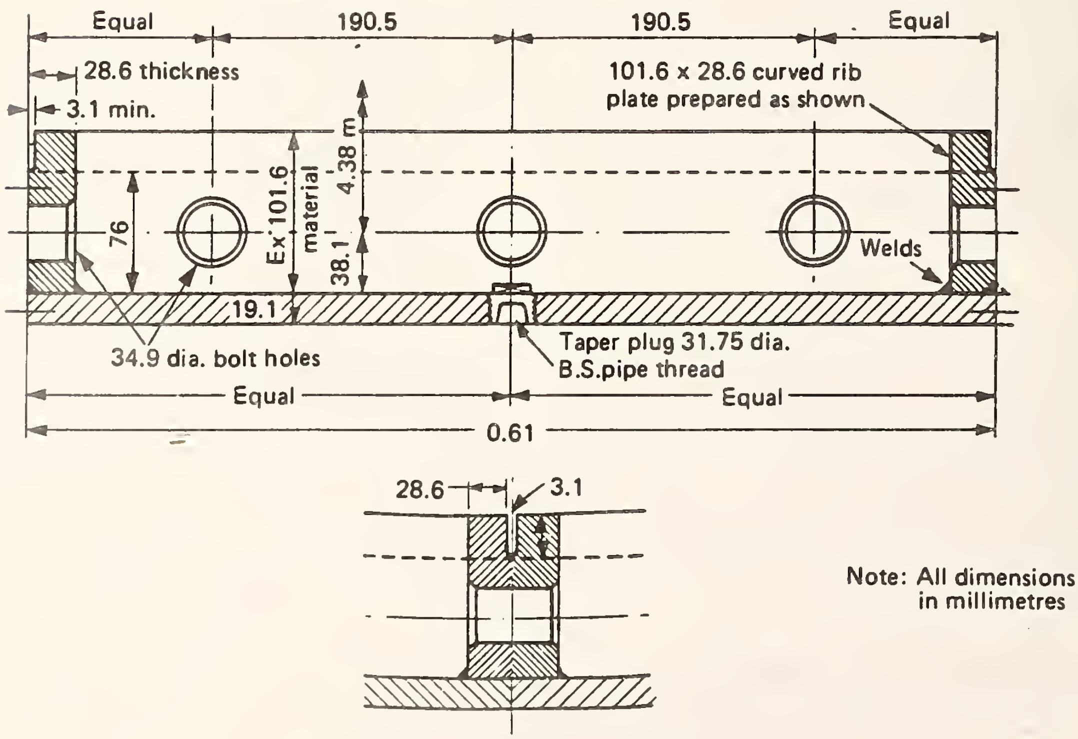


Figure 66 - TYPICAL CHANNEL SECTION SEGMENTS
(From Birkmyer, Ref. 17)



Detail of Dungeness 'A' tunnel lining



Detail of Dungeness 'B' tunnel lining

Figure 67 - DUNGENESS TUNNEL LINING
(From TRRL Report 335,
Ref. 34)

Gasket seals may be fairly firm and preformed or soft with adhesive qualities. Preformed gaskets of butyl rubber, cellular neoprene and polysulfide have been used. Gasket seals, whether formed or plastic, are usually considered a primary sealant to be followed by caulking. The soft plastic seals are more subject to damage and contamination by dirt under adverse tunneling conditions. A variety of current and proposed preformed gasket seals are illustrated in Figure 68 through 71.

5.50 GROUTING

5.51 Chemical Grouting for Cut-And-Cover and Soft Ground Tunnels

In the fluid state chemical grouts can penetrate into very fine voids and cracks in soil and rock where particulate grouts cannot enter.

In addition to controlling groundwater, chemical grout changes the engineering properties of the ground mass with little disturbance to existing structures. This, is of particular importance when excavating in urban areas.

In Europe a careful soil investigation followed by a well-planned grouting program is the most common primary means of groundwater control during tunneling in soft grounds; compressed air is used as a last resort. The grouting programs envisioned during design are left sufficiently flexible to allow the grouting contractor to plan the actual procedures to be used. In the United States, grouting is usually used only in problem areas where water cannot be controlled by other more conventional means.

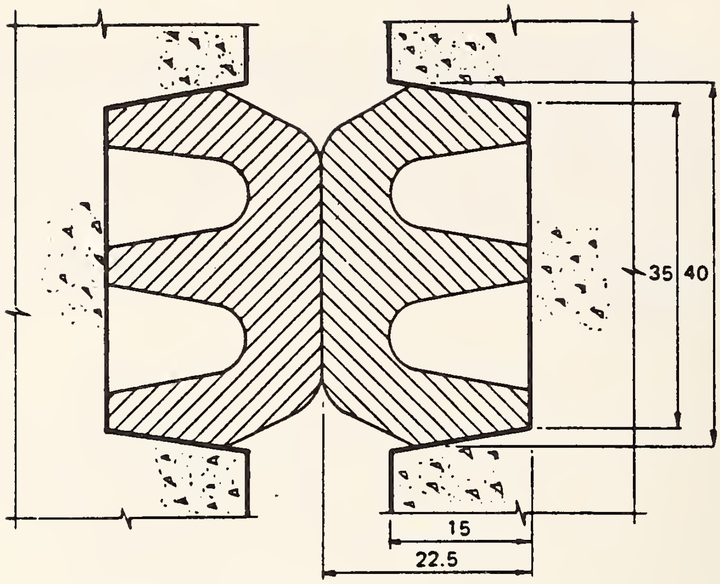
If the voids in the soil are sufficiently large, an initial grouting is performed using particulate grouts such as cement or bentonite. Then a second grouting with non-particulate chemical grout is used to seal the small voids so that the resulting product is essentially impervious. Grouting after excavation is not considered as effective as grouting ahead of the tunnel face.

The grout selection should be done by an expert in grouting. Field testing is important because of the numerous factors which influence the effectiveness of a grout. In fact, the gel time for a grout is frequently determined at the project site after field testing, so all influencing factors are included.

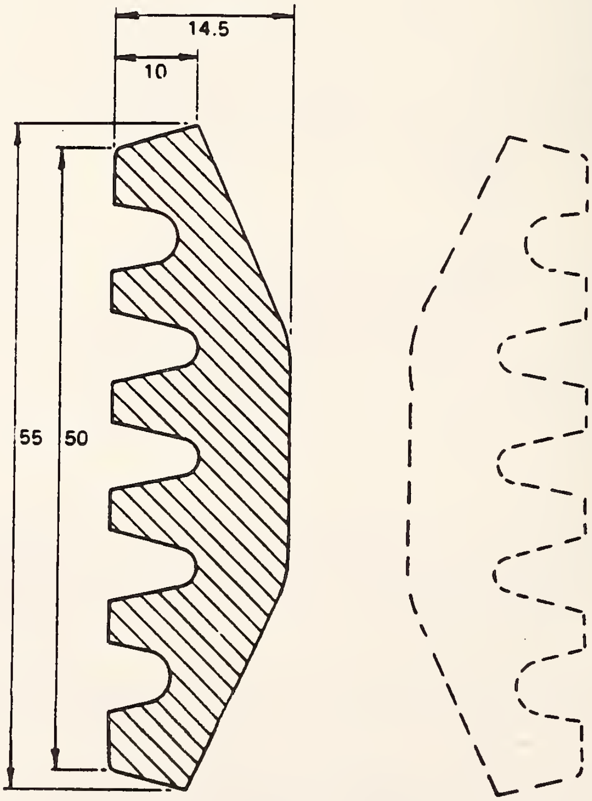
5.52 Consolidation Grouting of Rock Tunnels

Cement grout is the most inexpensive and most commonly used material for grout sealing of leaks through rock fissures (consolidation grouting) and filling voids between a cast-in-place

Proposed flexible joint gasket between bolted concrete segments, Ahmed Hamdi Tunnel, Egypt (Courtesy Sir William Halcrow and Partners)

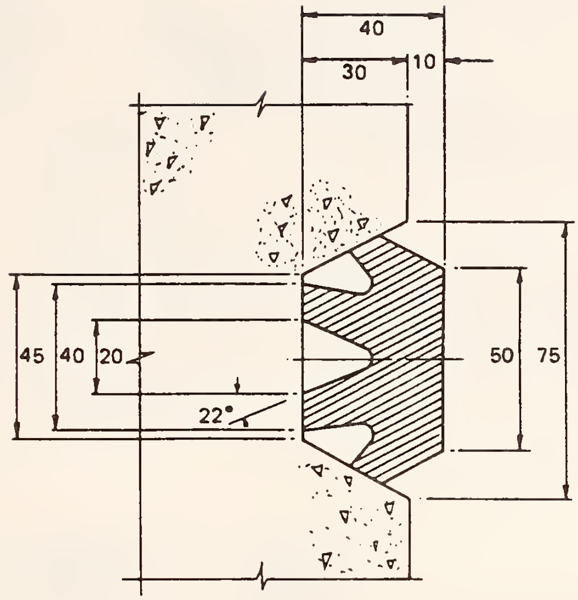


NOTE: Units in mm.



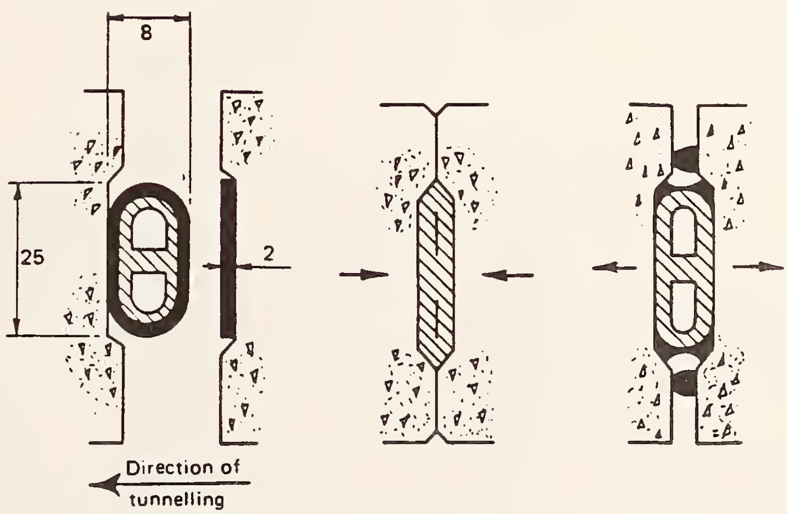
Joint gasket between bolted concrete segments, Isar River Tunnel, Munich, West Germany

Figure 68 - Sample Concrete Segment Gaskets (Egypt, West Germany) (From CIRIA Report 81, Ref. 25)



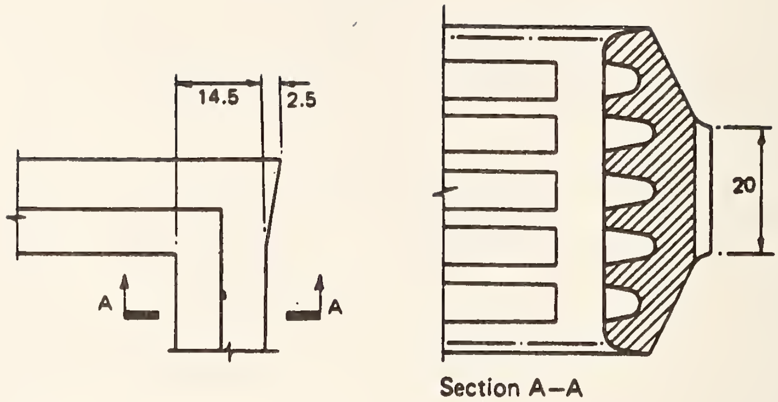
*Proposed joint gasket for concrete segments, West Germany (*Courtesy *STUVA)*

NOTE: Units in mm.



Proposed flexible joint gasket between concrete segments, Japan

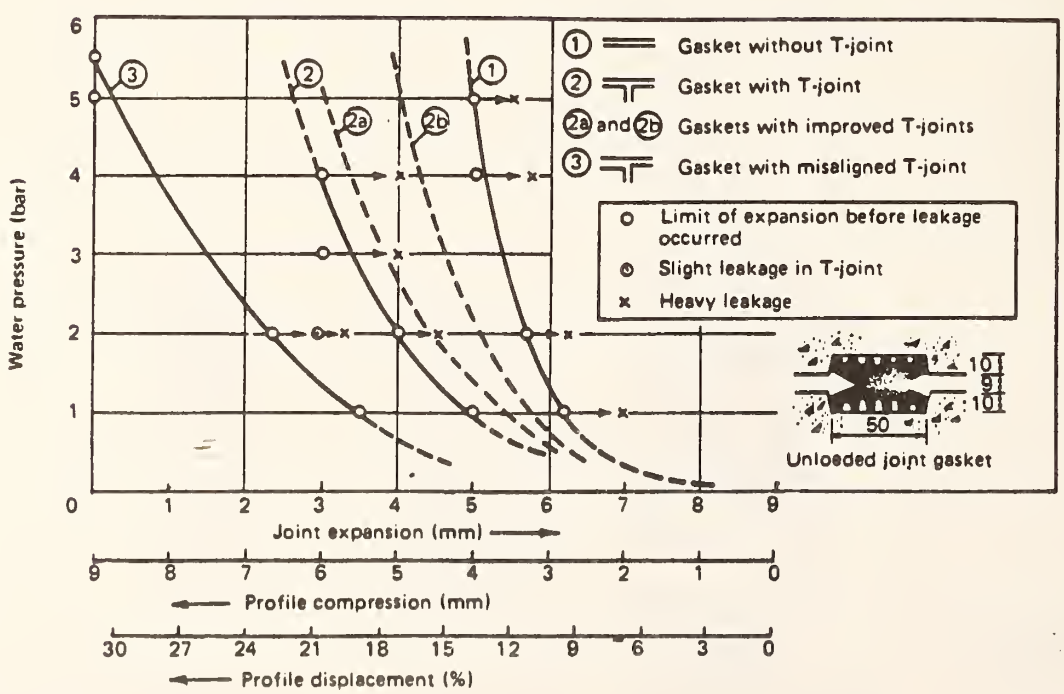
Figure 69 - Sample Concrete Segment Gaskets (W. Germany, Japan)
(From CIRIA Report 81, Ref. 25)



Joint gasket between bolted concrete segments, with improved corner detail, West Germany

(Courtesy STUVA)

NOTE: Units in mm.

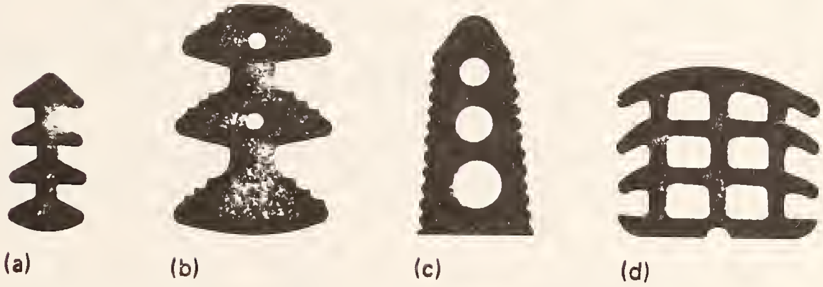


Results of tests on joint gaskets, West Germany (Courtesy STUVA)

Figure 70 - Sample Concrete Segment Gaskets (W. Germany) (From CIRIA Report 81, Ref. 25).

Leakage duct gasket, UK (Courtesy Colebrand Ltd)

Experimental leakage duct gaskets, West Germany (Courtesy STUVA)



NOTE: Units in mm.

Proposed leakage duct gasket, West Germany (Courtesy STUVA)

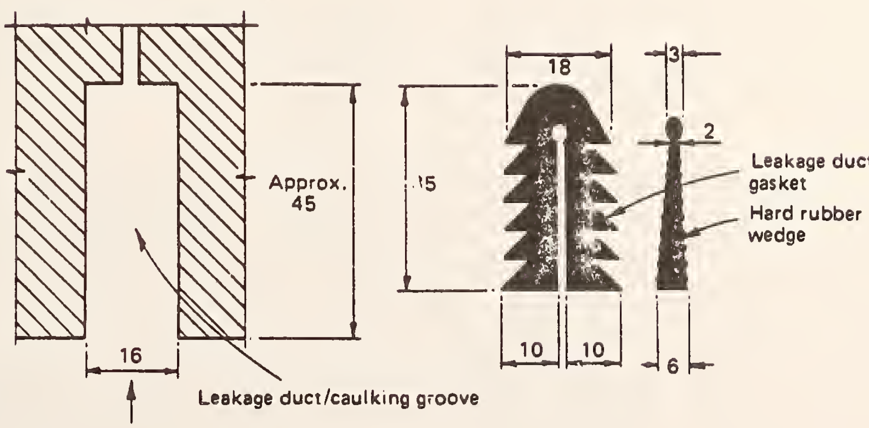


Figure 71 - Sample Concrete Segment Gaskets (Duct Gaskets)

lining and the excavated rock (contact grouting). In most rock tunnels the concrete lining and grout are the only measures taken to prevent water inflow. In unlined railroad tunnels, powerhouses or underground storage caverns, consolidation grouting may be the sole deterrent to water inflow.

Sample rock tunnels constructed for various agencies in the last decade, showing the project specifications for grout mixes and grouting equipment, are listed in Table 15. All of the sample tunnel specifications listed call for cement grouts as is common in most rock tunnels.

5.53 Annular Space Grouting in Soft Ground Tunnels

Most bolted linings are not expandable and consequently, a void of at least 2 to 3 inches (50 to 75 mm) remains around the circumference. To prevent settlement of the lining and the ground above, it is imperative that this void be filled. Traditionally, this has been done by blowing pea gravel (3/8 inch or 10 mm maximum) through the lining under the invert up the sides and over the arch. This is done with a gravel pan and air hose attached to threaded holes in the lining. Back from the heading 50 to 100 feet (15 to 30m) cement grout is pumped out into the void through the same holes.

Any sealing of water flows is a secondary benefit to the pea-gravel and grout and cannot be reliably controlled. Any grout that is taken by voids around the tunnel will reduce the permeability of the surrounding grounds, but it cannot be considered a trustworthy waterproofing method.

5.54 Contact Grouting of Concrete Linings in Soft Ground Tunnels

After the concrete lining is in place, it is necessary to grout any remaining voids between the primary and secondary linings.

The primary lining, whether it be ribs and lagging, liner plates, or segments, is not a smooth inside surface against which to place concrete. Therefore, no matter how conscientious the contractor may be, it is difficult to fill all voids when pumping concrete into a long steel form, even with form doors and vibrators. The most difficult area to seal is the arch where air can become trapped above the concrete between ribs or segment flanges. These voids must later be filled with a sand-cement grout. Eliminating these voids by filling them with grout is a definite aid to promoting watertightness of the final lining.

Table 15. SAMPLE TUNNEL CONSOLIDATION GROUTING SPECIFICATIONS

Tunnel	Owner	Date	Pipe Size	Equipment Cap.	Pumping Pressure	Mix Design, Etc.
Warm Springs Dam & Lake Sonoma Outlet Works Sonoma County California, USA	U.S. Corps of Engineers	1973	1-1/2" Dia.	Pump: Positive Disp. Similar to 'Moyno' Mixer, Sump, Hoses, Valves, Gages, etc.	Max of 30 psi at collar of hole	Cement and water with proportions to be approved. Sand in some cases only.
Thompson Yarra Tunnel Thompson River Development Melbourne, Australia	Melbourne & Metropolitan Board of Works	1968	As directed	Max. 400 psi Pump: Duplex piston type	As directed	Cement and water as directed. Sand if necessary.
Angeles Tunnel California Aqueduct Los Angeles County California, USA	Department of Water Resources, State of California	1966	By Engineer	Pump: Moyno Mixer: Harris Krough 50 GPM at 250 psi to 300 psi	Up to 100 psi	Portland cement and water.
Glendora Tunnel Foothill Feeder Los Angeles, California, USA	The Metropolitan Water District of Southern California	1965	1-1/2" Dia. Holes	As required to maintain service pressure capability not less than 1500 psi	Service pressures up to 1500 psi	As approved by Engineer. Mixture of Portland cement, calcium chloride, sawdust, or other approved material and water.
White Rock Tunnel Upper American River Project California, USA	Sacramento Municipal Utility District, Sacramento, California	1964	Min. Dia. 1-1/2"	Max of 200 psi Pump, Mixer, Hoses, Gages, valves, etc.	As directed	Cement and water.
Homestake Tunnel Lake and Pitkin Counties Colorado, USA	City of Aurora and City of Colorado Springs, Colorado	1963	1-1/2" Dia. Black Steel Pipe Std. Wt. (Sch. 40) ASTM A-120	Pump: Moyno Mixer, Agitator Tanks, Hoses: 1-1/2" Dia.	As directed	Cement and water as directed.
Upper Rubicon Dams, Tunnels, Access Roads, Etc. Upper American River Project California, USA	Sacramento Municipal Utility District, Sacramento, California	1962	Min. Dia. of 1-1/2"	Max of 200 psi Pump: Duplex piston type Mixer: Etc.	As directed	Cement and water.
Wachusett-Marlborough Tunnel Marlborough, Massachusetts USA of Massachusetts	Metropolitan District Commission, Commonwealth	1958	Up to 2-1/2"	As approved by Engineer.	Up to a max. of 100 psi to 300 psi as required.	Cement and water as directed.
West Delaware Tunnel Delaware System Delaware County New York, USA	Board of Water Supply, City of New York	1955	1-1/2" to 2-1/2" Dia. Steel	As approved by Engineer.	Up to 100 psi for large spaces. Up to 500 psi for small spaces.	Cement and sand as directed.

1" = 25.4 mm.; 1 psi = kPa

Two common methods of providing grout holes through the lining are placement of short sections of pipe behind the forms prior to concreting, or drilling of grout holes through the concrete. Grout holes are staggered with one hole per 50 to 150 sq. ft. (5 to 15 m^2).

Grout is pumped into a hole with just enough pressure to make it flow. When it reaches the next open grout pipe, the first is closed and the grout hose moved to the second. If a hole does not take any grout at the maximum pressure, usually under 30 psi (207 kPa), it is considered full. Care must be taken not to use too high a pressue to avoid cracking the concrete lining.

A low pressure grout pump, either a positive displacement position pump or a progressive helical cavity (Moyno) pump, can be used to place the grout.

5.55 Contact Grouting of Concrete Linings in Rock Tunnels

Contact (or backfill) grouting of concrete linings in rock tunnels is similar to that described for soft ground tunnels. Where lagging, blocking or cribbing are required with steel ribs, the probability for leaving voids increases. In such cases, it is advisable to place a grout pipe from the area to the form to insure that the area is not missed during grouting. In extremely bad ground where continuous lagging is required, it is important to place grout pipes through the lagging to insure grouting any voids between lagging and rock as well as between concrete and lagging.

The location of potential problem areas should be carefully recorded to aid in the subsequent placement of drill holes through the lining. As in the case of soft ground tunnels, it is important to fill all voids with grout to eliminate possible water courses behind the lining.

Table 16 summarizes contact grouting specifications for several tunnels in the United States and in Australia.

5.60 CUT OFFS

While steel sheet piling and diaphragm cutoff walls are often used for temporarily controlling groundwater during construction, they remain in the ground for the life of the structure. It is appropriate, therefore, to review the effects of such walls on the permanent structure in terms of groundwater control. Key factors to consider with respect to cut off walls are summarized in Section 4.30.

Table 16. SAMPLE TUNNEL CONTACT GROUTING SPECIFICATIONS

Tunnel	Owner	Date	Pipe Size	Equipment Cap.	Pumping Pressure	Mix Design, Etc.
Pacheco Tunnel - Reach 2 Central Valley Project California, USA	U.S. Dept. of Interior Bureau of Reclamation	1976	By Contracting Officer	Pump: Duplex Piston Type, Helical Screw Rotor Type, with 150 psi press capacity	Not more than 30 psi	Either of: 1. Cement and water 2. Cement, sand and water. Add Bentonite 2% by wt. of cement
Buckskin Mountains Tunnel Central Arizona Project Arizona, USA	U.S. Dept. of Interior Bureau of Reclamation	1974	By Contracting Officer	Pump: Duplex Piston Type, Helical Screw Rotor Type etc., with 150 psi press capacity	Not more than 30 psi	Either of: 1. Cement and water 2. Cement, sand and water. Add Bentonite 2% by wt. of cement
Warm Springs Dam & Lake Sonoma Outlet Works Sonoma County California, USA	U.S. Corps of Engineers	1973	1-1/2" Dia.	Pump: Air driven 15 GPM or more slurry capacity. Mixer, Sump, Hoses, Valves, Gauges, etc.	Max. of 30 psi at collar of hole	Either of: 1. 1 part cement and 2 parts min. filler and sand, (proportions to be approved). And when sand not possible: 2. Cement, water and mineral filler (fly ash)
Tonner Tunnels No. 1 & No. 2 Yorba Linda Feeder Los Angeles, California, USA	The Metropolitan Water District of Southern California	1972	2" Dia. Black Steel Pipe Std. Wt. (Sch. 40) ASTM A-120-69	As approved by Engineer	As directed by Engineer but not more than 50 psi	Either of: 1. Portland cement and water. 2. Portland cement, sand and water. Mix proportion to be determined by Engineer. Either of above to contain 2% of Bentonite by wt. of cement.
San Fernando Tunnel Foothill Feeder Los Angeles, California, USA	The Metropolitan Water District of Southern California	1969	1-1/2" & 2" Dia. Black Steel Pipe, Std. Wt. (Sch. 40) ASTM A-120-65	As approved by the Engineer	As directed by Engineer but not more than 50 psi	Either of: 1. Portland cement and water 2. Portland cement, sand and water. Either of above to contain 2% of Bentonite by wt. cement. Mix of proportion to be determined by Engineer.
Thompson Yarra Tunnel Thompson River Development Melbourne, Australia	Melbourne & Metropolitan Board of Works	1968	As directed Black Steel Pipe	Pump: As approved Mixer: As approved	25 psi or lower as directed	Cement, sand and water as directed.
Castaic No. 1, Castaic No. 2 Saugus and Placerita Tunnels Foothill Feeder Los Angeles, California, USA	The Metropolitan Water District of Southern California	1966	1-1/2" & 2" Dia. Black Steel Pipe, Std. Wt. (Sch. 40) ASTM A-120	As approved by the Engineer	As directed by Engineer but not more than 50 psi	Either of: 1. Portland cement and water 2. Portland cement, sand and water. Either of above to contain 2% of Bentonite by wt. of cement. Mix proportion to be determined by Engineer.
Angeles Tunnel California Aqueduct Los Angeles County California, USA	Department of Water Resources, State of California	1966	By Engineer	Pump: Moyno Mixer: Harris Krough 50 GPM at 250 psi to 300 psi	Max. of 30 psi	Water, cement, Pozzolan and sand with 2% Bentonite by wt. of cement

1" = 25.4 mm.; 1 psi = 6.9 kPa

5.70 SUNKEN TUBE TUNNELS

Present-day steel shells are machine welded construction which constitute a continuous membrane that is the primary waterproofing. A concrete lining is placed inside the steel shell under adverse conditions similar to lining a driven tunnel, which can result in a concrete lining of questionable waterproofing value.

The most common method for sealing the joints between the units is for divers to place a U-plate cofferdam spanning the joint as shown in Figure 72. Tremie concrete is placed between the cofferdam and steel shell completely encassing the joint. The space between bulkheads is then dewatered and steel plates are welded across the joint gap to make the shell/membrane continuous; concrete is then placed to make the lining continuous. Rubber gaskets have also been used as primary seals on some recent sunken tube tunnels.

Reinforced concrete tube units are usually covered with a steel or multi-ply asphaltic membrane, or a combination of the two.

If a steel membrane is used, it consists of a relatively thin welded plate, about 1/4 inch (6 mm), and is used as the base slab and outer wallform. After the roof is poured, the steel membrane can be continued across the top or a multi-ply membrane may be used for the roof and covered with a thin layer, 3 inches (75 mm), of protective concrete.

If ply membrane is used on the outer walls, timber protection will most likely be used.

In addition to the outer membrane, considerable care is exercised to make the structural concrete as dense and impervious as possible. In the Netherlands, where a number of such tunnels have been constructed, the procedures for producing impervious concrete have been sufficiently perfected that for several recent tunnels, the outer membrane has been eliminated.

Many special procedures are required including embedment of cooling water pipes, using low heat of hydration cement and use of wood as an insulator on forms to control heat shrinkage cracking.

To insure watertightness of the joint between slab and walls, a waterstop is used and the cold joint is rough chipped and grouted with cement mortar.

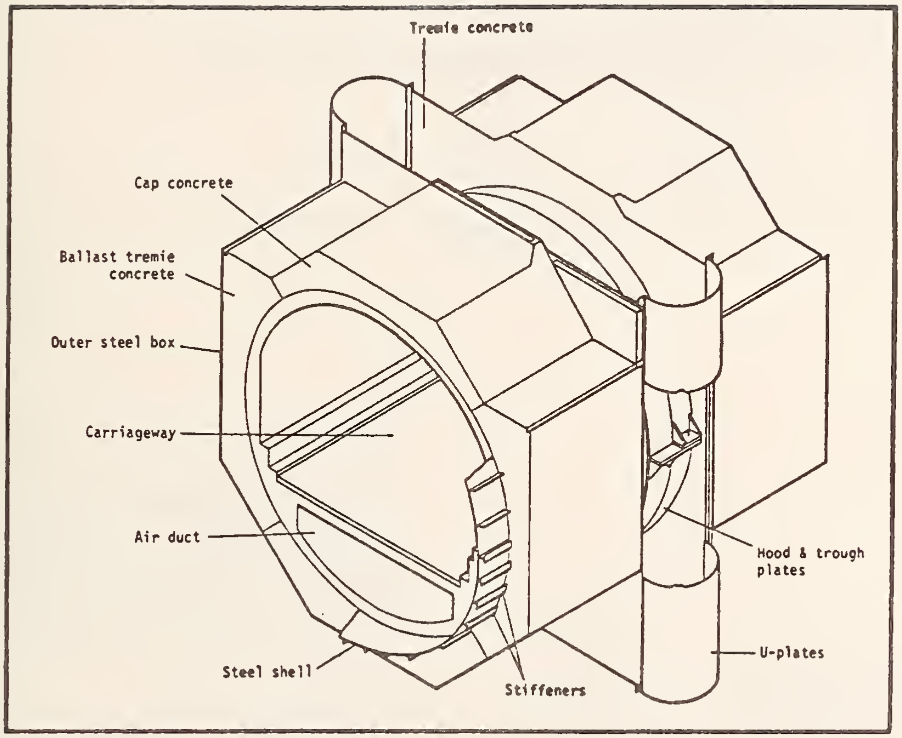
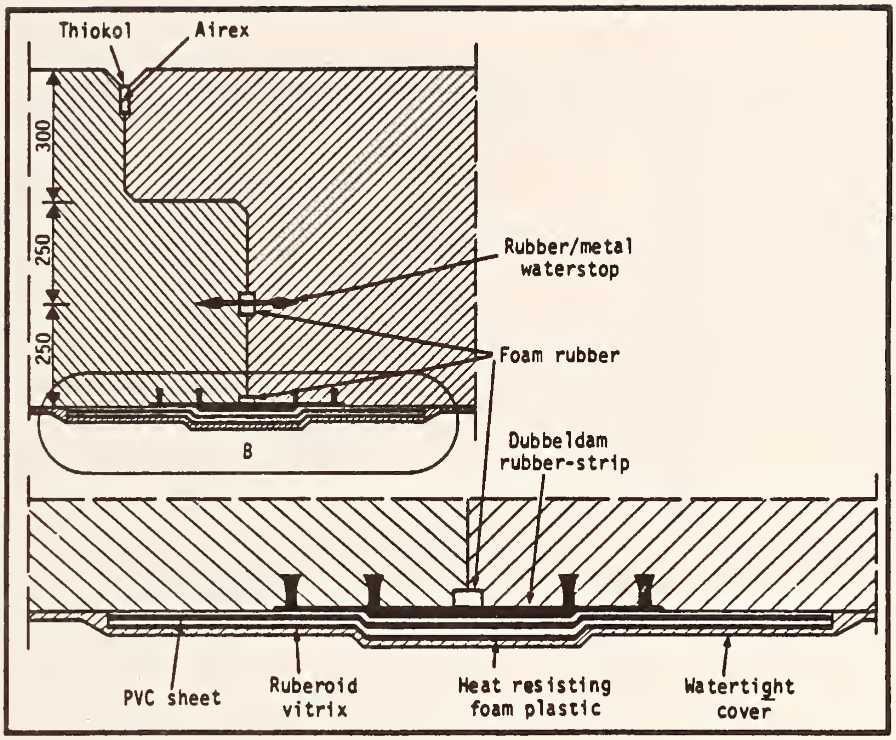
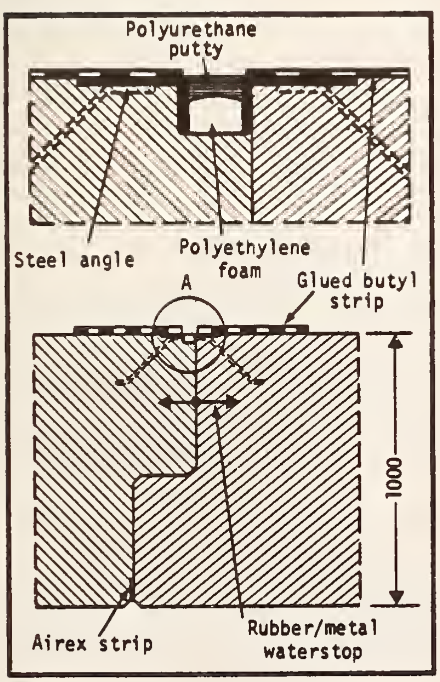


Figure 72 - ISOMETRIC VIEW OF A CIRCULAR STEEL-SHELL UNIT AND JOINT
(From Culverwell, Ref. 38)

See Figure 73, Details A and B, for joint sealing techniques. The joints between the sunken tube units are usually sealed with gaskets as shown in Detail D of Figure 74.



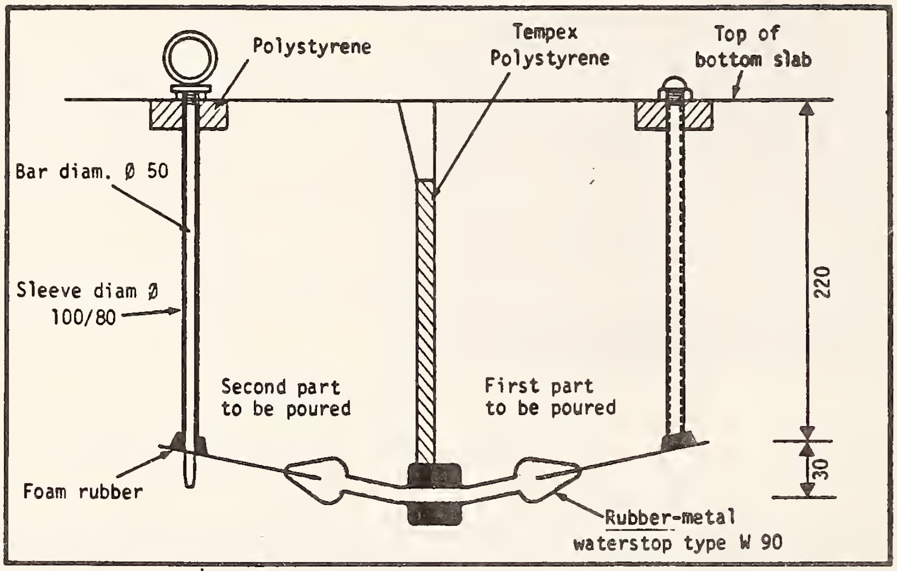
Detail A - EXPANSION JOINT WITH DUBBELDAM RUBBER STRIP



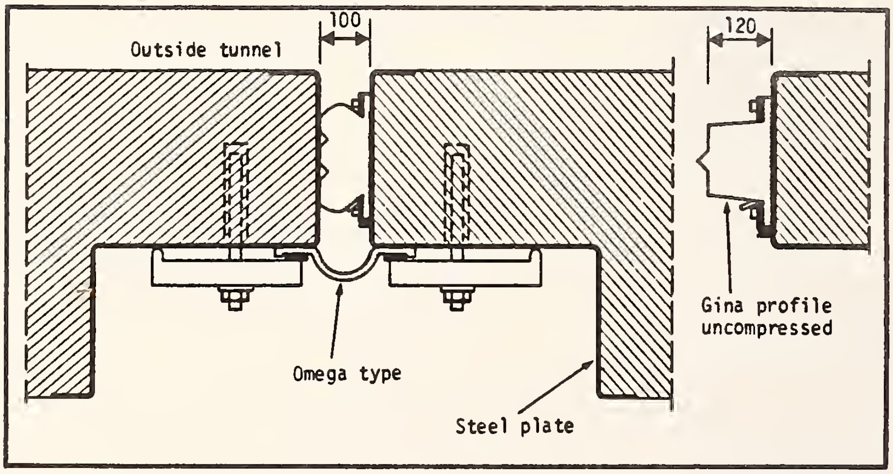
Detail B - EXPANSION JOINT WITH RUBBER/METAL WATERSTOP AND POLYURETHANE PUTTY

Note: All dimensions are in millimetres

Figure 73 - Joint Sealing Details for Concrete Box Tubes (Waterstops) (From Janssen, Ref. 73)



Detail C - DETAIL OF INJECTION TUBES TO RUBBER/ METAL WATERSTOP



Detail D - DETAIL OF UNIT JOINT SEAL SHOWING GINA GASKET AND OMEGA GASKET

Note: All dimensions are in millimetres

Figure 74 - Joint Sealing Details for Concrete Box Tubes (Gaskets) (From Janssen, Ref. 73)

6.00 MAINTENANCE, AND MONITORING CONSIDERATIONS DURING CONSTRUCTION

6.10 MAINTENANCE CONSIDERATIONS

Maintenance during construction is a significant consideration primarily for the active control methods, i.e., dewatering, recharge, freezing, compressed air and slurry and earth pressure balance shields. These all involve electro-mechanical and hydraulic systems which require routine preventive maintenance. Such preventive maintenance is provided by contractors and discussion of details is therefore not included in this report.

Encrustation is often a problem in dewatering systems that requires continuous maintenance to control. Maintenance techniques include; simple back flushing of the system, acid treatment, and removal of screens, wellpoints or ejectors for cleaning or replacement.

6.20 MONITORING OF DEWATERING SYSTEMS

With nearly all groundwater control methods some physical changes may occur in the adjacent soil mass. These changes may be obvious and desired as in the case of pre-drainage, grouting or freezing. On the other hand, unanticipated changes may occur such as surface settlement or heave, lateral movement (during construction of slurry walls or trenches), or ground loss and subsequent settlement (from compressed air of slurry shields). In either case, it is quite important that the engineer and/or contractor be aware of the conditions existing adjacent to the excavation. The knowledge obtained from monitoring these conditions is useful in determining the effectiveness of the dewatering scheme and detecting small problems before they become critical.

The majority of monitoring schemes associated with tunneling are designed to measure ground movements and stresses associates with the physical construction of the tunnel. Where dewatering has a significant impact on the success of the project, the important parameters to monitor may be groundwater levels as well as ground movements. In special cases, instrumentation of unique ground conditions may become necessary to determine the performance of the dewatering scheme (e.g., thermistors in ground freezing).

It becomes apparent that monitoring systems can become a vital part of the success of a dewatering system in sensitive urban environments. A well-planned system can reinforce the engineer's confidence with respect to the dewatering system design or identify its' shortcomings. In extreme instances potential problems can be averted before they develop.

The removal of groundwater from the soil mass along the sides of the excavation is accomplished by pre-drainage using wells, wellpoints, ejectors or drainage from within using sump pumps. The parameters critical to successful dewatering are obviously the amount and rate of drawdown. In urban areas the settlements associated with changes in effective stress may also be important.

Groundwater levels are easily measured using observation wells installed at predetermined distances from the tunnel. The rate of drawdown is verified by time-drawdown measurements in the observation wells and pumping rates in the extraction well. Excess pore pressures which develop due to construction activities can be monitored using piezometers. Groundwater observation wells are especially critical in areas where recharge systems are needed to protect adjacent structures.

Ground movements associated with pre-drainage techniques are a result of changes in effective stresses as pore pressures are reduced. In extreme instances these movements may be compounded by soil migration toward the wells or into the excavation (when sumping is used). Ground movements at the surface are easily measured by optical surveys. Deep movements are measured using deep settlement anchors (vertical movements) or inclinometers (lateral movements). Table 6 is a summary of commonly used monitoring instruments.

Some monitoring considerations relative to specific dewatering systems are:

Wells

1. A detailed construction log of each hole, showing depths to top of screen, bottom of seal, bottom of pump, etc., should be maintained.

2. For analyzing future problems, it is good practice to install a piezometer in the filter column and place a small (1" (25 mm) diameter) tube inside the well to measure operating levels. Flow meters are useful in monitoring system performance and detecting any slow deterioration.

3. Piezometers and observation wells should be installed along the alignment to indicate potential water problems before the tunnel reaches that point.

4. Records should be kept of all pumping rates, operating levels, and piezometer readings for both performance monitoring and detection of possible deterioration.

Wellpoints and Ejectors

1. All header connections should be vacuum tight. Vacuum gages should be installed along the header to help find air leaks. When all well-points are shutoff, the vacuum reading at the end of the system should be nearly the same as at the pump station.

2. A valve should be installed immediately in front of the pump station to check the pumps. In cases where a vacuum tank is used, the valves should be on each header that comes into the tank and vacuum gages installed both on top of the can and at the junction with each header line.

3. Valves should be installed along the wellpoint header, in order to isolate sections of the system if they become damaged.

4. Piezometers should be installed in the center of open excavations to indicate when drawdown is sufficient to allow excavation to begin.

5. Pressure gages should be installed along both header lines for both performance monitoring and to give clues to system deterioration.

6. Ejector systems should be checked to insure that each ejector is capable of pumping the flow available or that required to obtain drawdown.

7. Check ejectors to insure that the return flow is greater than the supply flow.

8. A flowmeter in the discharge is useful for detecting a slow deterioration of the systems.

9. Drawdown should be monitored by piezometers and the readings recorded for reference in detecting possible problems.

6.30 MONITORING OF RECHARGE SYSTEMS

Factors to be monitored during operation of a recharge system include:

1. Air bubbles can reduce soil permeability, therefore care should be taken to avoid aeration of the recharge water. Air should be vented at positive pressure points, where it will tend to accumulate. The conductor pipe should extend below water level to prevent turbulance.

2. The recharge water should be free of particulate matter. Sedimentation tanks are often used for this purpose.

3. The recharge water may have to be treated to prevent excessive encrustation. See Table 5.

4. Flowmeters, adjustment values, and pressure gages should be installed on each well to regulate inflows.

5. Excessive well head pressures should be avoided so that surface leakage and boils do not occur.

6. Piezometers should be installed to monitor system performance against operating criteria such as maximum allowable groundwater levels.

6.40 MONITORING CUTOFFS

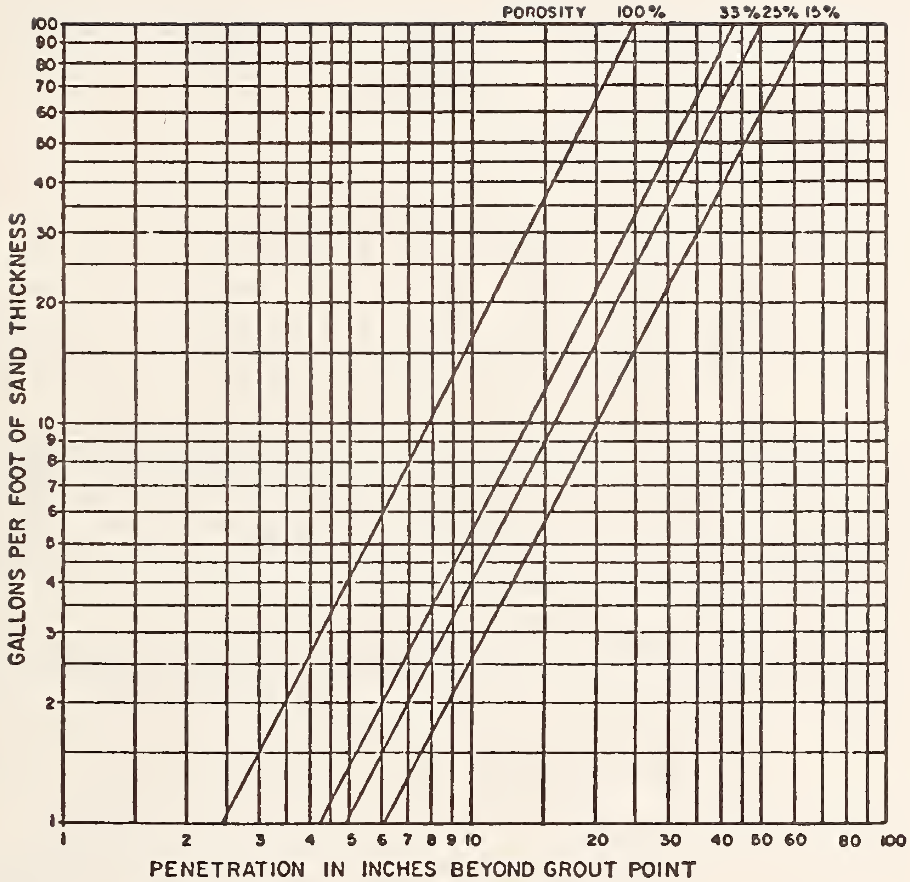
When impervious cutoffs are used to exclude groundwater, piezometers are used to insure that water levels are maintained outside of the excavation and those within the excavation are maintained below tunnel invert. In addition, wells and piezometers can be used to indicate zones of cutoff leakage. Lateral ground movement and settlement associated with cutoff methods are monitored most commonly by optical survey, however, the instrument systems listed in Table 6 are also commonly used. While ground movement is a significant concern, groundwater level information is the most important data for evaluation of cutoff effectiveness.

6.50 MONITORING OF GROUTING SYSTEMS

Prior to the start of grouting, guidelines must be set which would indicate that the grouting program is successful. Factors which should be monitored include:

1. Accurate records of grouting pressures, quantities and pumping rates to evaluate the progress of the program. Correlations can be made with theoretical estimates of soil voids to determine the extent of the grout penetration (see Figure 75). Several techniques for monitoring grout migration have been experimented with in recent years including seismic refraction, electrical resistivity, and accoustic emission monitoring. All of these indirect methods are still in developmental stages.

2. Grouting migration is also monitored by direct observation including test borings and test pits.



(1 in. = 25.4mm.)
(1 gal/ft = 0.01242 m^3/m)

Figure 75 - Grouting Volume Required to Fill Pore Space Radially Around Grout Point

3. Pressure testing of grouted zones can be used to evaluate the effectiveness of the program. These tests can also be used to provide in-situ permeability after grouting is completed.

4. Adjacent structures may be monitored to insure that the grouting process does not cause heave or lateral movement in foundations. Significant movement should serve as a warning to reduce grout pressures or terminate grouting altogether. Figure 76 illustrates guidelines for grouting pressures in rock.

6.60 MONITORING FREEZING SYSTEMS

1. Freezing is a specialized dewatering method and thus requires a unique monitoring approach. Prior to design of the freezing system, it is necessary to know if groundwater flows exist and, if so, how fast and extensive the flows are. This can be determined from observation wells or possibly acoustic emission techniques.

2. Soil temperature within the developing and completed freeze should be monitored to determine when the frozen structure is "closed" and excavation can begin, and to assure that the system is maintaining an adequate frozen zone during excavation.

3. Changes in groundwater levels inside and outside the developing frozen structure must be monitored for a proper elevation of performance. In shafts requiring frozen bottoms, a piezometer inside of the frozen shaft will rise as the bottom is frozen and thickens.

4. Surface and subsurface ground movements should be monitored for early warning of potential distress to adjacent structures from heave during freezing and settlement during thaw.

5. The coolant pressure, volume and flow should be monitored to assure that piping is unobstructed, leaks have not formed in the system and pumps are operating properly.

6.70 MONITORING COMPRESSED AIR, SLURRY AND EARTH PRESSURE BALANCE SHIELD METHODS

1. Monitoring techniques for compressed air shields, slurry and earch pressure balance shields typically involve measurement of ground movements associated

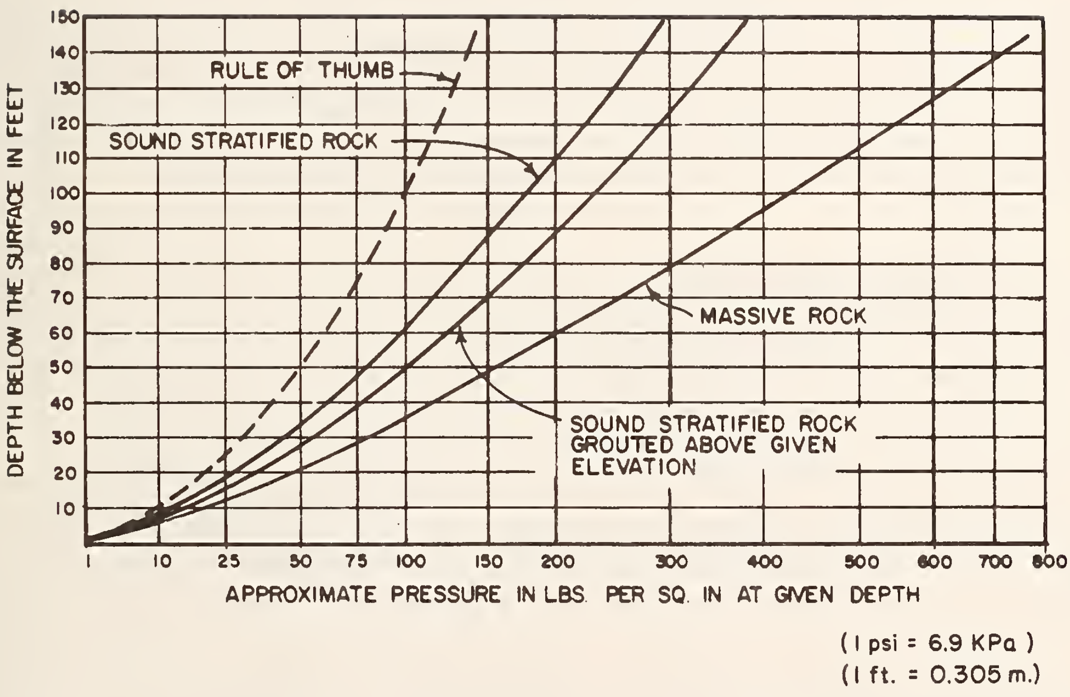


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FIGURE 76 - Guidelines for Grouting
Pressure in Rock (Ref. 81)

with the tunneling procedure rather than the dewatering technique. Water levels should be monitored along the tunnel alignment to determine compressed air or slurry pressure requirements. These measurements are very important where pre-drainage is used in conjunction with tunneling methods to reduce required air or slurry pressures.

2. When working under compressed air, safety is a major concern and therefore, it is very important to monitor air pressure, air loss and general operation of the compressor. A standby compressor is normally maintained so that in the event of any malfunction of the primary unit, tunnel stability can be maintained.

3. Slurry and earth pressure balance shields have some of the most sophisticated monitoring equipment of any groundwater control method. Most of it, however, has to do with system operation rather than evaluation of groundwater control effectiveness. Slurry pressures and densities are now commonly monitored with electronic process control equipment. For evaluation of tunnel method effectiveness. Groundwater levels and ground movement are still the most meaningful parameters to measure.

6.80 MONITORING ELECTRO-OSMOSIS

1. Monitoring of electro-osmotic flow can be accomplished using strategically placed piezometers to verify that the direction of seepage has been reversed. Pumping rates at the cathode wellpoints can also be monitored. Since electro-osmosis is typically employed in finer grained soils which have moderate to high compressibilities, settlements due to consolidation should be considered in the system design; the settlement can be monitored using conventional techniques previously described.

2. System operation is also observed including power consumption and electrode corrosion.

7.00 MAINTENANCE OF GROUNDWATER CONTROL SYSTEMS IN COMPLETED TUNNELS

7.10 DESIGN AND CONSTRUCTION PROCEDURES TO REDUCE MAINTENANCE

Any of the waterproofing systems previously discussed in this report could be successfully used alone if appropriate for the underground conditions and construction methods, and if ideally applied. "Ideal" application is usually not achieved because working conditions underground are not ideal, materials do not always meet specifications, and supervision of the work is less than perfect. Therefore, for a "completely dry" tunnel, a backup waterproofing system should be included in the design. This is particularly advisable if the structure will be at considerable depth, in a highly pervius aquifer, or covered with surface structures.

Providing two waterproofing barriers to the groundwater increases the cost and the time of construction. Rigid specifications and capable supervision also increase cost and construction time. Therefore, the consideration of less stringent construction controls with a backup waterproofing system added should be examined.

7.20 NORMAL MAINTENANCE PROGRAM

The heart of the normal maintenance program for the watertightness of a tunnel is efficient monitoring procedures based on regular inspection. If no program is established, workers inspecting or maintaining track, will probably not report water until it inconveniences them in their work.

7.21 Preventive Maintenance

Preventive maintenance is particularly valuable and necessary in bolted segmented linings. Bolts are not corrosion-resistant except in unusual circumstances. They are expected to corrode and replacement will be necessary. The routine inspection reports should include comments on the conditions of the bolts, nuts, washers, and grommets. This should be done by spot checking with special comment on those items near areas where leakage is occurring. Then, at the convenient time, a work crew could replace the bolts, nuts, washers, and grommets in the deteriorating areas and reseat the caulking, if necessary, preventing water infiltration.

Caulking in the joints may also be monitored in a similar way.

If monitoring is not done, a regular maintenance crew should periodically patrol the tunnel correcting sealant deficiencies as they are noticed.

Another item for preventive maintenance inspection is the interior coating of fabricated steel segments, if exposed. If this coating deteriorates, the steel is exposed to the corrosive environment of the tunnel and over a period of many years could deteriorate to the point of failure. Re-coating need not be done frequently, but when large areas show damage to the interior protective coating, the coating should be restored.

7.22 Monitoring Leakage

In general, it is impractical to repair every small moisture intrusion or minor drip when first observed. It is usually more economical to wait until a number of small leaks or a major leak is noted. Frequently, small leaks heal by solidification of material carried by the seeping water.

Transportation tunnels are customarily inspected routinely for general condition and operational adequacy. During these routine inspections, any sign of water leakage should be described in the insepction report. Of course, if the flow is large, immediate notification of the maintenance manager should be made. Also noted in the report should be the absence or presence of soil in the water.

The amount of flow in litres/day or gallons per hour is difficult to estimate without considerable experience. The inspectors who would observe leakage while performing their other inspecting duties may not be trained for that sort of determination. For this reason, a scale of descriptive leakage should be established to give a certain uniformity to the reports of all inspectors. This could be similar to that prepared by CIRIA and quoted in Section 1 of Volume 2. These should be keyed to common experience. A suggested list is given.

1. Visible shiny water film, possibly dripping slightly, affecting an area less than 1m

2. Constant seepage from a narrow crack

3. Sheet flow from area less than 1m

4. Sheet flow from area more than 1m

5. Flow more than a faucet leak but less than garden hose flow

6. Crack or joint flow equivalent to moderate garden hose flow or more. Notify maintenance manager immediately.

Particular attention should be paid to leaks that increase since this indicates increasing deterioration of the waterproofing system and the possibility of imminent structural damage. Any indication of soil in the flow indicates loss of ground and possible strength decreases. This decrease in strength may result in a shift in load distribution on the tunnel lining producing stresses unanticipated at the time of design. A decrease in flow may indicate that the groundwater was depositing soil particles or precipitated chemicals in the cracks and pores and the integrity of the watertightness might eventually be restored.

Consecutive inspection reports should be compared as they are received and if an individual leak becomes significantly larger, a new large leak appears, or if leaks in one area are becoming more numerous or have remained about the same size for a long period of time, maintenance procedures should be started and all significant leaks and leaks of long duration should be repaired. Of coarse, if a large leak is reported that needs immediate attention, consideration should be given to repairing all leaks at that time.

Another means of determining the presence of significant leakage into a tunnel is to maintain a record of the output of the sump pumps. If dry weather output is significant, it is obvious that water is flowing into the area of the tunnel that feeds the pump. If no washing or other water producing work is being done in that area, it must be checked for leaks. This program is likely to flag only large individual leaks or numerous smaller ones. The drainage system, including the sump pumps, has been designed for anticipated maximum flows from water entering through stations and tunnel entrances during inclement weather. If pump output exceeds this appreciably, the tunnel should be checked for leakage.

7.23 Repair Methods

One repair method that is useful in many situations is to inject particulate or chemical grout into the ground surrounding the tunnel. The characteristics of the soil determine the most useful type of grout, as discussed in Section 5.50. A grouting system is then chosen, holes are drilled through the leaking areas into the ground mass, and grout is injected until the leaking stops as described in Section 5.22. Usually there will be enough information from the tunnel construction phase that

a group of grouting systems for routine maintenance can be established for the major formations surrounding the tunnel. These would be used unless leakage is catastrophic or cannot be controlled by the specified system. Then it would be necessary to resort to the assistance of a grouting expert and a soil investigation.

The applied waterproofing membranes that are in sheet form cannot normally be replaced or repaired under wet conditions. For these materials, it is generally best to inject grout behind the waterproofing if soil conditions permit. Unfortunately, the location of the leak in the membrane may be some distance from the appearance of water within the tunnel. The water may travel long distances between the waterproofing membrane and the structural or inner tunnel lining before leaking through a porous place in the inner lining. Therefore, several injections may be needed before the leakage is eliminated.

The maintenance of bentonite envelope waterproofing is usually not difficult. Small leaks heal themselves by the movement of the bentonite gel into the cracks through which the water flows. For major leaks, bentonite slurry is injected through as hole drilled at the location of the leak. Then the drilled hole can be sealed with a hydraulic cement plug and the tunnel will again be leak-free.

One of the advantages of cementitious waterproof coatings is that leaks, if they occur, are localized right where they come through. They can be repaired easily by a reapplication of the waterproofing coating on the cleaned and roughened existing surface even though wet.

Where chemical or particulate grouting of the ground mass has been used as the primary groundwater control method, regrouting will usually suffice to stop the leak.

If the means of controlling groundwater is permanent lowering of the groundwater table by drainage, and leakage occurs, the drainage system must be checked and if flow is lower than its capacity, any clogging of the system must be cleaned out. If the flow is near full capacity, a source of unanticipated water supply must be sought and controlled by some other means.

If the interior caulking between segments loosens, all that is required is that it be reinstalled. Since this caulking is most frequently lead, a small air hammer does the job. Some additional lead may be required to make the joint completely watertight again.

In sunken tubes, the section joints are the most probable locations for leaks. These are usually inaccessible since the exterior is usually covered by a clay or concrete blanket and the interior by concrete lining or architectural finish, and the grouting of the blanket is the most usable means of restoring watertightness.

7.30 UNUSUAL MAINTENANCE

Unusual maintenance is required when neglect has caused major deterioration of the tunnel, differential settlement has occurred, the lining has been damaged by a major fire in the tunnel, or there has been some other occurrence which jeopardizes the integrity of the tunnel.

When a fire occurs in a tunnel, it may produce cracks or porosity in the tunnel lining material; warp or crack metal segmented linings, open the seams or destroy the integrity of the waterproofing over an extensive area without a collapse of the tunnel. Portions of the tunnel linings that have been damaged must be replaced. If the integrity of the waterproofing has been breached, it would probably be difficult or impractical to replace. Extensive grouting of the soil outside may be necessary.

If deterioration of the tunnel lining has progressed until there is such strength loss that the lining fails structurally or if earth movements cause major cracks or breaks in the lining, water inflow will be the maximum which can flow through the exposed ground formation. "Maintenance" then becomes major structural repair. To accomplish this work, the tunnel must be dewatered. This can frequently be done using the same methods as were used during the construction period.

If the means of preventing water infiltration during construction was compressed air, the damaged section can be cut off from the rest of the tunnel with temporary bulkheads and pressurized until the restoration work has been completed. If slurry walls were used as water control, it is sometimes possible to box the area around the damaged tunnel with slurry walls or slurry cutoff walls, dewater, replace the original slurry wall with formed concrete, and replace the interior tunnel finish. Sheet piles, especially if grouted, can be used in lieu of slurry wall. If the tunnel had been built by cut-and-cover methods it may be possible to reach the affected area from ground level. Local ground freezing may be employed under appropriate conditions even though it may not have been used in the original construction.

A versatile tool for stopping the flow of groundwater into a damaged tunnel is quick-setting injection grout.

Construction procedures for rebuilding the damaged section would be similar to the construction procedures used for the original tunnel.

8.00 LEGAL AND CONTRACTUAL CONSIDERATIONS

8.10 CONTRACT DOCUMENT

The objective of contract documents for a tunneling project should be to achieve a desired result at minimum cost for the project through the proper sharing of the risks associated with groundwater control.

8.11 Basic Elements

The documents provide the contractual framework for tunnel construction, set the context for price determination as well as for the execution of the work. The contract documents are the primary tool for control of contingency costs, especially with regard to groundwater control. The means by which this control can be effected through contract documents include the following:

1. Disclosure of all subsurface exploration data, both factual and interpretive. This should be done, whatever the scope of the exploratory and testing programs. The contractor's ability to anticipate groundwater control problems is directly related to the amount of information available to him. Further, the cost of the project is only increased if the contractor must gather his own information in order to duplicate data already obtained, but withheld from bidders, by the Owner.

2. A clear statement of the responsibility of the contractor for groundwater control should be couched in terms of practical, realistic results to be achieved. Further, there should be a clear definition of the means of measurement by which the achievement of these results will be determined.

3. There should be a minimum limitation placed on the bidders' ingenuity to produce the specified results. There should be procedural provisions to exclude unskilled or inadequate consideration of groundwater problems in bidding and to demonstrate in advance of the tunneling that system designs are competent, that systems performance is adequate, and that appropriate monitoring will be done as the work progresses.

4. Payment provisions should allow for fair compensation for groundwater control activities, but without an incentive to unnecessarily expand this element of the work, such as through unit prices for quantity of water or grout pumped or for additional wells.

5. There should be a change mechanism for accomodating unanticipated aspects of groundwater control. Because monitoring of the dewatering activity can provide early warning of such conditions, it is an important consideration in controlling the adverse effects of changed groundwater circumstances. For these reasons, an owner designed and operated groundwater monitoring system may prove to be a good investment in minimizing these effects.

6. Whether the change mechanism is through a changed--condition clause, or some other procedural feature of the contract, the contract documents should clearly describe its features. The amount of price increase in bids caused by contingencies will be directly influenced by how well the bidders understand how unanticipated groundwater conditions will be identified and how the costs of their resolution will be compensated.

8.12 Execution

Once it is decided how much of a groundwater control design will be specified and how much will be left to the option of the contractor, it is necessary to accurately convey this information in the contract documents. The specificatons must be written in a way which clearly state what is required of the contractor for performing groundwater control. The specifications should also convey how the performance of this groundwater control system will be measured. By knowing what is exactly required of him and also upon what basis the performance of the groundwater control system will be measured, the contractor will have the best chance of designing a system which will perform as specified.

Sometimes such a partial specification takes the form of the Owner soliciting a lump sum price bid for additional wells as they may be required. The procedure has some merit, but in practice difficulties often developed. In variable soils the number of wells required is often a function of the contractor's skill in conditions encountered, and in selecting the most favorable sites for wells. It is not in the Owner's interest to have given the contractor an incentive to construct as many wells as possible.

An alternative procedure is for the Owner to design and specify a minimum dewatering system that the contractor must install. The responsibility for the adequacy of the system, and the cost of any supplementary effort required, remains with the contractor. The advantages claimed for the method are several. It assures that a reasonable dewatering effort will be made in advance of excavation. In the course of installing the minimum system, an experienced contractor can develop data to help him gage the necessity of supplemental work. The minimum system approach avoids the confusing division of responsibility for groundwater control and it also reduces the possibility that an inexperienced contractor will attempt the work with unsuitable methods.

If part of the groundwater control system is specified, it is necessary that the anticipated performance of the system is accurately spelled out, e.g. definition of groundwater levels to be acheived or working conditions. The contractor must know the extent of groundwater control to be provided by the design system, and how much additional control he is responsible for maintaining. By eliminating any ambiguity and overlap of responsibility, contingencies supporting this uncertainy can be eliminated from the bid price.

When conditions indicate that more than one groundwater control method is capable of providing the satisfactory result, a groundwater control method should not be specified in the contract documents. If a groundwater control method is specified, a statement to the effect that alternate methods of groundwater control are allowed should be included. This allows for ingenuity in groundwater control and also produces the lowest competitive bid. In this case, it is necessary to state exactly what the groundwater control system is required to do and also how its performance will be measured. If the contractor is allowed to design his own groundwater control system, full disclosure of all technical information should be contained in the contract documents.

One of the objectives of specifications is to protect the Owner from unrealistic pricing of bids as a result of inadequate or unskilled approaches to groundwater control. A procedure which has proven successful in this regard is the two-stage submittal. Prior to beginning work, the contractor submits to the engineer for review a detailed plan of his proposed dewatering system. Review by the engineer does not relieve the contractor of his responsibility for the adequacy of the system, but it does provide the engineer with an indication of the thoroughness with which the contractor is anticipating the control of groundwater. During the construction of the dewatering system the contractor, in accordance with good practice, will be making observations and conducting tests to evaluate the underground conditions.

This information will be much more complete than that available at the bid and may suggest substantial design modification of the final design. Hence, a second submittal is more meaningful. After completion of the dewatering system installation and prior to the start of tunneling, the ocntractor again submits a detailed plan of the dewatering system as constructed, together with test data and computations demonstrating that the system is capable of achieving the specified results. This second submittal therefore forces the discovery of pertinent supplementary information and through the demonstration of results in advance of tunneling, minimizes the risks of interrupted production and other costs associated with unanticipated groundwater problems.

8.20 ALLOCATION OF RISK

For the purpose of this discussion, risk is defined as the potential for harm, damage, or other adverse effect, all of which may be measured in terms of added cost of the toal tunneling projected. Avoidance of risk has, in the recent years, become an increaingly popular concept throughout our society and commerce. But some level of risks resides in any undertaking. Construction has been described as the most complex activity routinely undertaken by man. Risk is embodied in the natural environment of the construction activity and in the human procresses involved (principally communications). The complexity of the construction activity accentuates the risk inherent in communication (disputes, claims, etc.).

Tunneling is perhaps the most complex construction activity occurring in the underground, a high-risk environment. This peculiar concentration of risk potential is a forceful background for considering how to allocate equitable shares of the risk to the participants in a tunneling project.

A summation of the elements of current practice can be organized from this perspective, considering three principals in the tunneling enterprise (Owner, Engineer, Contractor) and dividing the potential risks into three topical categories (environmental or site-related, communications-related, and procedural) as follows:

8.21 Environmental

The site-related risks are generally considered to be the responsibility of the Owner. In othe words, the Owner assumes these risks when the tunnel alignment was determined. In groundwater terms, the risks involve the adverse effects of groundwater control problems and from the adverse effects of groundwater control upon third parties. The risk control techniques that have been found to be most effective are as follows:

1. A technically adequate investment in subsurface exploration prior to design and bidding;

2. Full disclosure of the results of subsurface explorations in the bidding documents

3. The provision of a change mechanism, such as a changed-conditions clause in the contract documents, perhaps with its effectiveness enhanced by an owner-designed monitoring system.

8.22 Communcations

Most of the risks associated with disputes over division of responsibility, interpretation of contractual provision, appropriateness of changes in procedure, etc., are fundamentally manifestations of communication deficiencies. After he has completed his technical design tasks, the Engineer's primary role is that of communicator. Hence this category of risks is best controlled by the Engineer through such devices as:

1. Disclosure of basic design intent and concepts.

2. Proper definitive specifications, as discussed in earlier sections of this report.

3. Review of the technical aspects of the Contractor's groundwater control program so as to insure not only technical adequacy but the thoroughness of consideration as well.

4. An effective program of continuous monitoring throughout the project.

8.23 Procedural

This category of risks is mostly closely related to the Contractor, whose proper role is to assess the groundwater problem and effect solutions within the limits of the data available, and hence to monitor groundwater conditions so as to anticipate the need for modifications before tunneling progress is affected. Accordingly, the risks generally fall into the category of increased costs associated with delays, and unreimbursable costs. These cost elements are often a second layer of cost added to the contingency prices already included in the bid items in order to accomodate uncertainty in the basic informaiton available at the time of bidding. The techniques available to the Contractor for the control of these risks are as follows:

1. A competent program of exploraton, testing and design so as to develop the least expensive and most flexible groundwater control system.

2. The conscientious pursuit of an adequate program of groundwater moritoring after the intial system has been installed.

3. The utilization of the contact provisions for changes in compensation in connection with unanticipated groundwater conditions.

From this perspective, it is evident that the concept of allocating risks must be based on two equally important concepts; who is in the most effective position to mitigate a particular variety of risk, as well as who is equitably responsible for the costs involved? The advantage of considering the concept from this viewpoint is to make more evident the fact that the process of risk allocation involves responsibility for action (mitigation) as well as reaction (compensation). Much has been done in consideration of the reactive aspects of the concept. However, it is likely that the total cost of the project would be significantly reduced if equal attention were addressed to the responsibility of the various parties to take action to control risk and to concentrate on providing incentives for such action.

8.30 SAMPLES OF CURRENT PRACTICE IN SPECIFYING CONTROL METHODS FOR PERMANENT STRUCTURES

To sample current practice of tunnel groundwater protection methods several dozen individual transportation and other tunnel contract specificiations were reviewed. Three tables have been prepared showing the basic type of structure, waterproofing and other control methods specified, maximum allowable leakage (where specified) and specified treatment of tunnel leakage. There is one table each for cut-and-cover structures, (Table 17) soft ground tunnel structures, (Table 18) and rock tunnel structures (Table 19). Tunnels from seven rapid transit systems comprise most of the entries, but tunnels from four other local and federal agencies are also presented. Each table is presented in reverse chronological order with the more recent projects listed first. Because of the amount of data and variety of water protection methods used, it was impractical to list all data in a single page format. Tunnel linings, water protection methods, and joint treatments are detailed on a second sheet of each table and letter-coded for use in the table, as applicable to each tunnel project. Where alternate protection methods are

allowed by specification, all are listed. Most agencies will permit substitution of contractor-proposed alternate protection methods provided the contractor and/or the material manufacturer can prove, through tests or performance records of previous installations, that it is equivalent to the method specified.

8.31 Cut-and-Cover Structures

Sample specification summaries of methods for controlling groundwater in cut-and-cover underground structures are given in Table 17. Although not specifically listed as a waterproofing method, many of the structures listed rely to a large extent on thick, good quality concrete walls and slabs as a primary deterrent to water intrusion. The required membrances are more of a secondary defense to keep water from any shrinkage cracks or other cracks that might develop. The concrete box structures are highly stable, water resistant structures if the concrete is placed properly and any minor defects are corrected.

The exception to this, as mentioned in Section 3, is the line section of the New York City Transit Authority. Built as a composite structure of structural steel bents at 5-foot (1.5 m) centers with unreinforced concrete between, this structure specification requires a stringent waterproofing membrane of brick in asphaltic mastic and built-up asphalt ply membrane. Although the concrete specifications are as exacting as other agencies, the steel-concrete interfaces are difficult to bond completely and allow possible leakage. Thus, it is necessary to rely on more severe waterproofing measures, which, though relatively expensive, have proved effective over many years of use.

The other waterproofing methods specified cover a range of membrane materials including built-up asphalt membrances, with or without reinforcing, butyl rubber, and bentonite panels. There is no apparent trend in the use of materials for the years covered and they more likely reflect the preference of the designers and the locations of the project. One WMATA specification requires merely dampproofing, and one on MARTA requires no waterproofing except at joints. Most require membranes on the roof or roof and sides only, relying on a heavy base slab, needed to counteract buoyancy to be sufficiently water resistant.

Most structures require PVC waterstop at joints, but some specify bentonite in niches with bentonite panels covering the outer concrete face at the joint. The Edmonton station specifications required a metal waterstop.

Only on the BART system were specific allowable leakage requirements given. All specifications called for leakage repair but only a few designated specific remedies of injecting bentonite slurry or epoxy.

8.32 Soft Ground Tunnel Structures

Sample soft ground tunnel specifications on groundwater control methods are given in Table 18. While the majority of cut-and-cover structures are cast-in-place reinforced concrete, this is not necessarily so in soft ground tunnels. There are a number of possible ground support/lining combinations, which may affect the type of waterproofing or sealing methods that can be used. Specified primary and secondary lining (and possible alternates) are listed in code as are the specified waterproofing measures. A list of code definitions is given on the second sheet of Table 18.

Of the seven tunnels listed five are rapid transit line tunnels and two are sewer main tunnels. Three have cast-in-place concrete linings specified, three have segmented linings specified, and the seventh has several alternate possible linings. The three tunnels with segmented linings are transit tunnels and have a partial concrete lining of invert and walkway only. In general, grouting is required outside the lining, and between the primary lining and concrete on projects where there is a full concrete lining.

Cast-in-place linings require PVC or copper waterstop at the joints although the contractor on the New York Subway project received approval to omit the copper waterstop and provide remedial grouting where required instead. Those projects calling for steel segmented linings required protective coatings inside and outside. Most segmented linings additionally required plastic grommets for connecting bolts and caulking between segments. On the Baltimore tunnel a joint sealant was specified in lieu of caulking.

Where the allowable leakage permitted in the transit tunnels was specified, it was comparable to those given for cut-and-cover structures in Table 17. The allowable leakage given for the San Francisco sewer tunnel was about eight times that allowed for transit tunnels. Leakage treatment requirements were also comparable to those for cut-and-cover tunnels, consisting of pressure grouting, bentonite slurry, or epoxy injection.

8.33 Rock Tunnel Structures

Table 19 gives sample specifications for groundwater control in completed rock tunnel structures. As in the case of driven soft ground tunnels, the primary and secondary linings of rock

tunnels may have an effect on the choice of groundwater control measures. While a number of the tunnels researched has alternate primary lining choices, they all specified cast-in-place concrete secondary linings. Though some segmented linings have been built in rock tunnels, there are very few and these have been discussed in Section 4 of this report.

All tunnels specified consolidation grouting of rock (where required) and contact grouting of the secondary lining. Most required waterstops for joints although some called for bituminous, sponge rubber, or paraplastic fillers. The Melbourne Underground Rail Loop Authority (MURLA) rapid transit projects required plastic grommets for bolts and caulking for those sections using steel liner plastes as a primary lining. The New York Subway tunnels in rock did not required any waterproofing for the inverts which were relatively thin slabs. Weep holes in the invert were specified with the tunnel drainage system designed to take the anticipated leakage.

The only tunnel specification in this group to designate an allowable leakage was the MURLA rapid transit. It is interesting to note that this requirement was considerably more severe than those given in U.S. rapid transit tunnels in the previous tables. Those specifications that gave particular requirements for leakage treatment mentioned grouting, repairing or replacing portions not watertight.

9.00 RECOMMENDED AREAS FOR ADDITIONAL STUDY

9.10 OVERVIEW

Most groundwater control methods employed during construction have been practiced for many years and are well understood. Possible exceptions to this general statement include the newer specialized soft ground shields, certain grouting techniques, and ground freezing. For groundwater control in completed tunnels, the basic methods, i.e. high quality concrete, grouting and some of the older applied membrane methods are well understood, however, certain technical details do require further study, such as use of the newer membrane materials and certain gasketing materials and details.

Groundwater control, both during construction and in completed structures, continues to be much of an art and this situation will probably continue for the foreseeable future. Advances will probably be made in small steps through method variations on individual projects and case history information will, therefore, be invaluable to the advancement of the state of practice. Engineers must be encouraged to publish detailed, yet concise, case histories of successful and unsuccessful applications of groundwater control methods. It is particularly important to publish those case histories where significant problems were encountered, and it could be said that first attempts at groundwater control failed.

During the course of this investigation, numerous individuals involved with groundwater control in the tunneling were interviewed. Based on these discussions plus the experience of the principal authors, areas for potentially fruitful additional study were identified.

9.20 SPECIALTY CONTRACTOR INVOLVEMENT IN DESIGN

Much groundwater control is practiced by specialty contractors who are not generally consulted in detail during the design process because of potential conflicts due to vested interests in a particular method. Detailed consultation with specialty contractors during the design process, including the development of project specifications and contract provisions, can be extremely helpful. It is believed that with the proper use of available expertise, significant savings and smoother running contracts can be realized. Detailed study of fair contractual mechanisms which permit a specialty contractor to participate in design, yet not be excluded from bidding, needs to be conducted, supplemented with case studies.

9.30 PROBLEM IDENTIFICATION

There also needs to be continuing study to make both designers and contractors aware of new, innovative groundwater control methods and what they can accomplish. Many smaller tunnel projects, i.e. tunnels on the order of 8 to 12 feet (2.4 to 3.6m) in diameter, are built by medium sized to small contractors who, rather than trying an innovative method, will fight the water with resultant loss of ground and surface settlement. Designers should identify anticipated groundwater problems in the contract documents and require the contractor to take a positive approach to the problem rather than just allowing him to fight it.

9.40 PRE-QUALIFICATION

There needs to be additional studies of the advisability of developing specification language which would prevent unqualified contractors from attempting tunnel constuction under difficult groundwater conditions, and which would also prevent unqualified subcontractors from attempting specialty techniques. Development of fair and equitable prequalification provisions can be difficult to achieve. It is felt that publication of case studies of where it has been attempted plus discussion with both engineers and contactors would prove enlightening in this regard.

9.50 SUBSURFACE EXPLORATION

Improved subsurface exploration is a continuing area of study in the tunnel industry. It is recommended that a study be considered to identify that information which is most meaningful to contractors for evaluation of groundwater control. It is not believed that major new exploratory techniques need to be studied, but rather the information which it typically provided to contractors can be reviewed and contractors interviewed in a systematic manner to identify a general perception of what information is most useful and whether or not there needs to be additional information of a particular kind provided.

9.60 IMPROVED GROUTING TECHNIQUES

Areas which are being studied in some detail in the grouting industry include development of more economical chemical grouts such as economical silica grouts plus monitoring of grout migration and the effectiveness of grouting programs. Several techniques have been tried, including various geophysical methods. One of the most promising at this time is the accoustic emission technique. It is suggested that this work continue and that it is an area for fruitful additional research.

9.70 APPLICABILITY OF SLURRY AND EARTH PRESSURE BALANCE SHIELDS

The new soft ground shields, i.e. slurry and earth pressure balance shields, have been used extensively in Japan and to a lesser degree in Germany, England, and the United States. While there has been considerable work done primarily by the Japanese on the development of this equipment, there is still some concern relative to evaluation of proper ground conditions under which each type of shield works most effectively. In particular, the optimum conditions for slurry or earth pressure balance shields are different and additional work is required to more clearly define these optimum conditions. This is an area for further study with much of it again being involved through case history information.

9.80 SUITABILITY OF GROUNDWATER CONTROL METHODS IN COMPLETED TUNNELS

This is an area where there is probably not a need for major new research or development of new techniques. Rather, the most productive area of study would be to conduct a systematic evaluation of these methods in common use today and their effectiveness. It is suggested that case studies of groundwater control methods commonly used in the United States be prepared and that the information be presented in a very concise format with proper references to allow readers to obtain specific details on each method if desired.

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

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Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

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0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.